CONTROL OF GROUNDWATER BY TRENCHING METHOD FOR SLOPE STABILITY OF GRANITE SOIL AREA

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CIVIL ENGINEERING
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Nurhidayah binti Shaari

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the requirements for the
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CERTIFICATION OF APPROVAL

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Approved by,

(AP Dr Nasiman Sapari)

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TRONOH, PERAK
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CERTIFICATION OF ORIGINALITY

This is to certify that I am responsible for the work submitted in this project, that the original work is my own except as specified in the references and acknowledgements, and that the original work contained herein have not been undertaken or done by unspecified sources or persons.

NURHIDAYAH BINTI SHAARI

ABSTRACT

The objective of this study is to find the influence of lowering groundwater level on slope stability. The project is examines soil moisture as a factor that influence the slope failures, soil characteristics, and the design of the control method for residual soil from granite. The slope stability issues concerning some factors that influenced the slope failures are investigated and presented. It focuses on granite soil because most landslides in Malaysia occur in granite soil. Residual soil in granitic area consists of minerals from the weathering products of feldspar, quartz and mica minerals. Soil are taken from Bukit Kledang and brought to Geotechnical Laboratory for tests. From the laboratory session, the soil is a fine soil containing 17.75% of moisture content during normal condition, and has acidic pH value of 4.71. The specific gravity of granite soil is 2.655. The liquid and plastic limit tests indicate that the plasticity index of the soil is 7.46%. The content meaning the soil is silt with low plasticity (ML) according to the plasticity chart. The soil also has a hydraulic conductivity, k from 0.001 to 0.01 cm/sec which still under the standard of fine sand. In addition, from the shear box test, the shear strength of the soil from the study area is 82.907-141.029 kN/m^2. The modification of the soil; by adding 5% of Calcium Hydroxide (CaOH) is not really affected the effectiveness of the water flow. Even though the modified soil has higher k value, immediate test shows that it has low shear strength. Trenching is an innovative technique to construct a protective trench at the slope area by using gravel and lined with geotextile. The design of trenching also consists of perforated pipes and drain to flow the groundwater out. Trenching is a good option to intercept groundwater which is perched above a relatively impermeable soil. Laboratory testing was conducted based on a trenching model for granite soil based on the design. From the tests conducted to the trenching model, it shows that the moisture content of the soil (15.96-21.22%) is less than the liquid limit (39.50%). It means the trenching lowers the moisture level. The water that flow out from the perforated pipe is also clear and without sediment. From the Slope/W analysis, the factor of safety determined is 1.454 in order to get the stable factor of safety as applied by Malaysia standard. This control method is considered to be cost effective because the materials to be used are confined to the trench only.

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CHAPTER 1
INTRODUCTION

1.1 Background of Study

Groundwater level is one of the factors affecting the stability of slope. The increment or high groundwater level will cause instability of slopes as the occurrence of water will reduce the strength of soil masses. It is therefore important to control the groundwater level in order to ensure the slope is stable. There are several causes that can increase the shear stress or decrease the shear strength of the soil mass including heavy rainfall, earthquake, pore water pressure from groundwater and many other human activities such as excavations but the main reason for landslide in Malaysia is mainly heavy rainfall (Brand, 1989). Slope failure formed by loosely compacted and completely decomposed granite occurs commonly during intense tropical rainstorms. The increases of groundwater level also induced the slope failure. To reduce this failure such as landslides, experts have come out with many types of slope stability control using agriculture and engineering controls.

1.2 Problem Statement

There are many cases of slope failure in Malaysia such as in Highland Towers, Bukit Antarabangsa, and Genting Sempah Highway landslides. The landslides in Malaysia are associated with heavy rains and the increment of groundwater level. The slope failures occur in many types of soil. The most critical soil is granite. Granite are typically has thick soil formation and contain substantial amount of clay. Many slope protections have been developed but slope failure remains a serious problem particularly to development on hill slopes. So this study will attempt to investigate the effectiveness of trenching as a slope stability control.

1.3 Objectives of Study

The objectives of the study are:

i) To determine the engineering properties and characteristics of granite soil.

ii) To determine the characteristics of the soil with and without modification.

iii) To design and evaluate the effectiveness of trenching in granite soil for slope stabilization.

1.4 Scope of Study

The scope of the study is associated with the slope stability of granite soil in Bukit Kledang, Perak.

The main points as the scope of study consist of the following:

i) Investigation of granite soil characteristics.

ii) Find the factors that cause the slope failure, the existing control and the problems associates with the control.

iii) Design and evaluate the effectiveness of trenching in granite soil for slope stabilization

1.5 Relevancy and Feasibility of the Project

This study will be relevant because Malaysia is facing a serious problems associated with slope stability. The existing controls still face a lot of weaknesses. That is why at certain area, even though there are a control there, but the landslide or slope failure still occur. In UTP also, there are some slope failures even though the slopes are provided with surface drainage and vegetation cover as controls. Thus, this study will provide an alternative for slope stabilization by trenching method. The trenching method has already been applied in some slope in Malaysia but its influence on safety parameters is yet to be ascertained. It is expected that this method is more reliable and cost effective. The author believes that this method will have it application in some slope in Malaysia.

CHAPTER 2
LITERATURE REVIEW AND THEORY

Review for the study was taken from some related books, journals and the internet. Basically, spot to be highlighted for the study consists of slope stability and the relationship to groundwater, the existing control and the fundamental of the control. Here are the reviews of the slope failure:

2.1 Theory

Slope is an exposed ground surface that stands an angle with the horizontal. The slope in soils or rocks can be a natural or man-made and ubiquitous in any structures. Slope is generally less expensive than constructing walls; so that many structures such as dams, highways, canals and stockpiles are constructed by slopping the lateral faces of the soil. Natural forces such as wind, water and snow may change the topography on Earth that can create instability in slope. It means the ground surface is not horizontal anymore and the gravity component will tend to move downward and if the gravity is large enough, slope failure can occur. Some slopes are gently rounded, while others are extremely steep.

Slope stability is based on the interplay between two types of forces, driving forces and resisting forces. Driving forces promote downslope movement of material, whereas resisting forces deter the movement. So, when driving forces overcome resisting forces, the slope is unstable and results in mass wasting or slope failure. Slope stability analysis is difficult to perform but it is very important to avoid the slope failure. The analysis can determine the shear stress developed along the most likely rupture surface and comparing it with the shear strength of the soil. The main purpose of slope stability analysis is to contribute to the safe and economic design of excavations, embankments, dams, and soil heaps. It concerned with identifying critical geological, material, environmental, and economic parameters that will affect certain project, as well as understanding the nature, magnitude, and frequency of potential slope problems (Abramson, et al., 2002). It is also based on simplifying assumptions, and design of stable slope relies heavily on experience and careful site investigation (Budhu, 2007). The factors of slope stability are material, strength of rock or soil, slope angle, climate, vegetation and time.

There are some basic principles involved in slope stability analysis such as factor of safety. Factor of safety is defined by the formula $F_s = \tau_f / \tau_d$ where F_s is the factor of safety with respect to strength, τ_f is average shear strength of the soil and τ_d is average shear stress developed along the potential failure surface to maintain stability. Safety factors provide a margin of safety to account for unquantifiable factors: temporary loads, such as high wind gusts, ice, and projectile impact; imperfect workmanship; manufacturing variations; and transportation, handling, and erection stresses. Some experts have called safety factors as 'factors of ignorance' (Wonneberger & Bortz). That is because safety factors are larger when loads and stresses are uncertain, when the material strength is highly variable, and when the material is not very forgiving. Safety factors are larger when the behaviour of the material and the loads are less known. From the equation $F_s = \tau_f / \tau_d$, it shows that the shear strength consists of two components, cohesion and friction and may be written as $\tau_f = c' + \sigma' \tan \varphi'$ where c' is the cohesion, φ' is the angle of friction and σ' is the normal stress on the potential failure surface. In similar manner, it can write as $\tau_d = c_d' + \sigma' \tan \varphi_d'$. c_d' and φ_d' are respectively the cohesion and the angle of friction that develop along the potential failure surface (Das, 2001). The failure is assumed to occur when F_s is less than 1 because it is in a state of impending failure (Das, 2001) but according to Budhu (2007), the usual range of factor of safety is 1.15 to 1.5. According to Barre (1987), the safety factor recommended by Stone Associations for granite is 3.0. Figure 2.1 shows the example of slope in front of Pocket C, UTP.

Figure 2.1 Example of Slope in front of Pocket C, UTP

2.2 Literature Review

2.2.1 Soil Properties of Granite

According to Hossain (1999), residual soils are widely distributed more than three-quarters or about 75% of the Peninsular Malaysia and a significant proportion of the soils is granite. Due to slow cooling, associated with the depth or pressure inside the earth's crust, residual soil in granitic area consists of minerals from the weathering product of large crystals of Quartz, Biotite Mica and Feldspar (Potassium feldspar and Plagioclase Feldspar) and other minor constituents. The percentage composition of feldspar varies between 65-90% of quartz can extend from 10 to 60% and that of biotite lies between 10 to 15%. Many steep slopes in these soils often have deep groundwater table above which the soils possess high matric suction (Hossain, 1999). It is important to study the shear strength characteristics of the residual soils particularly with respect to matric suction because the stability of natural or a cut slope in these soils depend on the shear strength. Granite is often by-products of mountain building. It is very resistant to weathering, frequently forms the core of eroded mountains. Weathering is the breakdown process of rock to form sediment. Granite undergoes physical and chemical weathering. During hydrolysis (chemical weathering), the process involves the dissolving of feldspar minerals in the granite by hydrogen. The feldspar reacts with hydrogen in water producing Kaolin (china clay) in the processes of Kaolinisation - this occurs as water circulates through the granite. Granite weathered by hydrolysis becomes weakened as the quartz crystals remain as loose crystals, unaffected by the hydrolysis process. Another products of the process are Sodium (Na) and Potassium (K) ions but these ions will be removed through leaching. The Biotite Mica will undergo hydrolysis to form clay and oxidation to form iron oxides while the quartz will remain as residual minerals because it is very resistant to weathering. Granite undergo physical weathering by frost shattering process. It results in granular disintegration and thawing of water in joints or crevices in the rock. Granite soil lies under silt to clay type. Other properties of granite are porosity and absorbency. Porosity is the ratio of pores (micro-voids) in the soil, to its total solid volume. Porosity ratio of granite is normally from 0.4-1.5%. Absorbency is an important determining factor in soil sensitivity to stains. Granite has 0.2 - 0.5 absorbency value. Both properties are not tested in the lab because no apparatus provided in the lab. So the value stated is the theoretical value only. From the Table 2.1 below, it shows that clay and silt has high erodibility value. That is why in granite area, it is easy for slope failure to occur.

Table 2.1 Slope Angle and Soil Type vs Erodibility

Slope Angle and Soil Type vs. Erodibility

Slope angle	Erodibility	Soil type
50%	Very high	Silt
40%		Silty sand
30%		Clayey sand
20%		Organic soil
15%		Clays
10%		Silty gravel
5%		Sand
<5%		Gravel
	Very Low	

2.2.2 Types of Slope

There are two types of slopes which are natural slope and man-made (engineered) slope. Natural slopes that have been stable for many years may suddenly fail because of changes in topography, seismicity, groundwater flows, loss of strength, stress changes and weathering. Some natural slopes have old slip surfaces; result from previous landslides or tectonics activities. The shearing strength along these slip surfaces often quite low because prior movement has caused slide resistance to peak and gradually reduce to residual values.

There are several types of man-made or engineered slope. These include embankments, cut slopes and retaining walls. The engineering properties of materials used in embankments and fills are controlled by the grain size distribution, the methods of construction, and the degree of compaction. The embankment slopes are designed using shear strength parameters. These embankments consist of cohesionless soils (sands and gravels), cohesive soils (silts and clays) and a mixture of both soils; gravels and cobbles (earth-rock mixtures). Organic soils, soft clays, and silts are usually avoided. The range of particle sizes of embankment fills is governed, for economic purpose; by the availability of the materials from nearby areas (Abramson, et al., 2002). Landfill is another man-made slope; it is special cases where both cut and fill slopes are involved and where the fill materials are much less than optimum. Landfills may contain organic materials, tree limbs, refuse, and variety of debris that commonly dumped, pushed and spread then compacted. The last man-made slope is retaining structures. It is used to support stable or unstable earth masses. There are different types of retaining structures including gravity walls, tieback or soil-nailed walls, sheet pile walls, and mechanically stabilized embankments including geosynthetics and georid reinforced walls. These structures are used in some principal ways such as external stability of the soil behind and below the structure, internal stability of the retained backfill and structural strength of retaining wall members.

2.2.3 Slope Failures and the Factors Induced

Slope failure, which include landslides or mass wasting, is the downslope movement of rock debris and soil in response to gravitational stresses. It is a phenomenon that slope collapse abruptly due to weakened self-retainability of the earth under the influence of a rainfall or earthquake. This sudden collapse may cause many of people fail to escape from their place, thus resulting in a high number of fatalities. It depends on the soil type, soil stratification, groundwater, seepage and slope geometry.

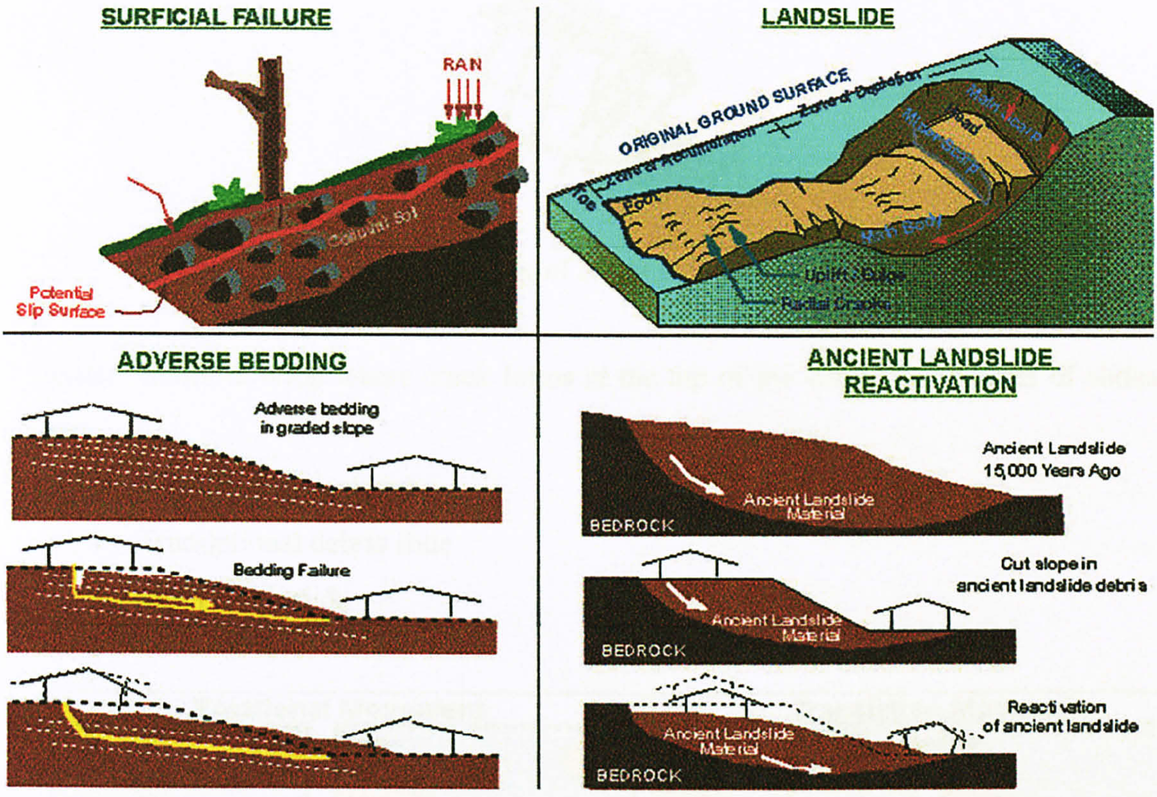


Figure 2.2 Slope Failure

Based on the work by Vanes 1978, slope failure or landslide can be classified into five main groups; falls, topples, slides, spreads and flows.

Falls: Rock or soil detaches from the slope and move rapidly to its new resting place. Often associates with undercut cliffs and riverbanks

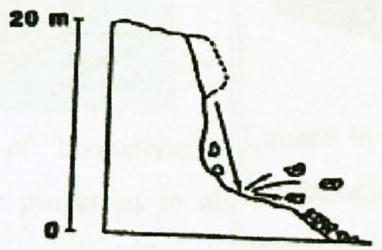


Figure 2.3 Type of Slope Failure; Falls

Topples: Topples involve rock or soil that tilts or rotate forward on a pivot point. There are no necessary much movement but it may lead to falls or slides of displaced material.

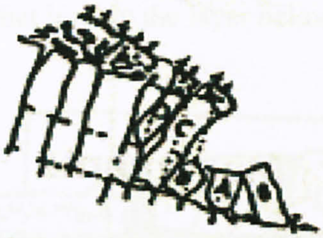
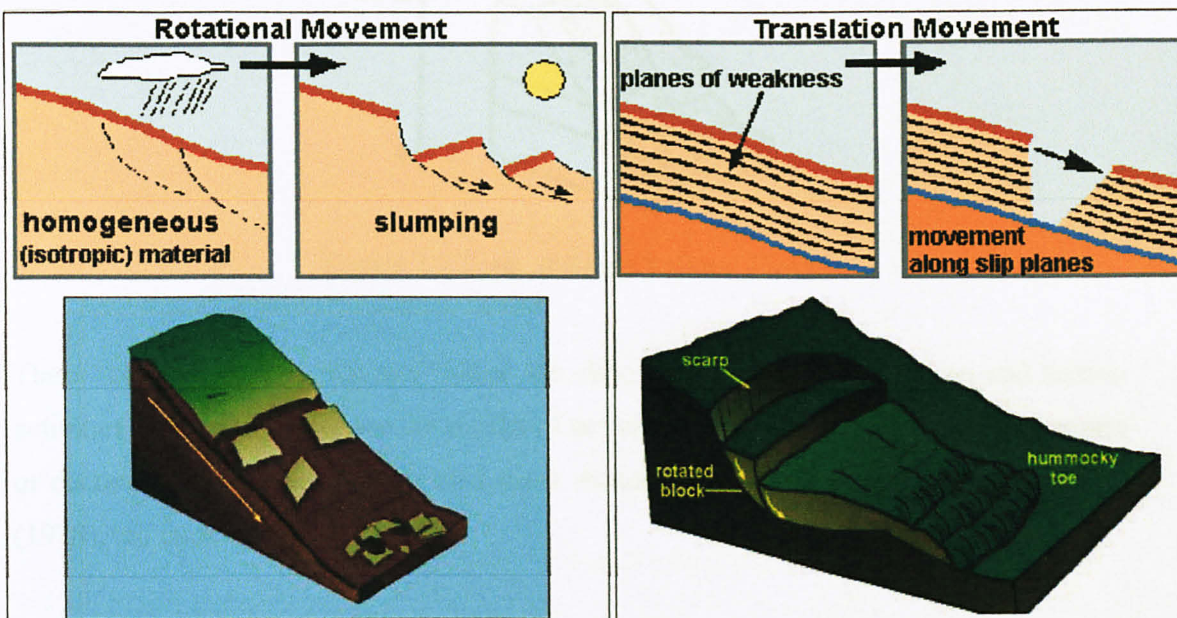


Figure 2.4 Type of Slope Failure; Topples

Slides: Slides develop where crack forms at the top of the slope. Three types of slides are:

- Rotational rock slump
- Translational debris slide
- Earth block slide



Caused by slope failure of homogeneous (isotropic) materials (similar properties in all directions).	Caused by slope failure of inhomogeneous (anisotropic) material with planes of weakness.

Figure 2.5 Rotational Movement and Translation Movement of Slides

Spreads: These landslides involve sudden horizontal movement on very gentle terrain. It is often initiated by earthquakes that liquefy the layer below the moving material.

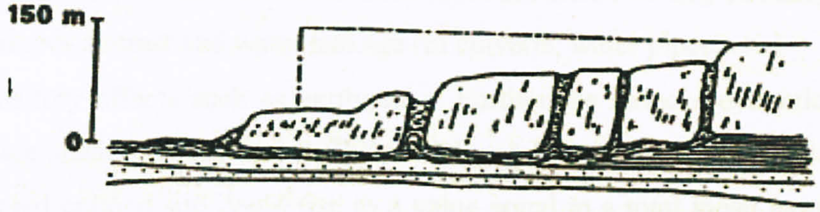


Figure 2.6 Type of Slope Failure; Spread

Flows: Flows are the downslope movement of unconsolidated material in which the material behaves like a viscous fluid. Flows can be very slow or can be exceedingly fast. There are four rules apply for flows which are earthflow, mudflow, debris flow and debris avalanches.

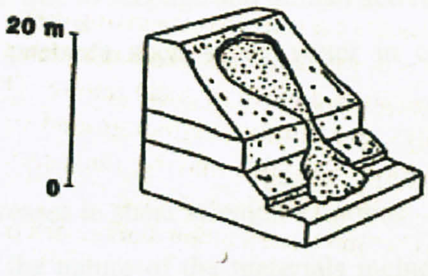


Figure 2.7 Type of Slope Failure; Flow

There are many causes that can induce the slide. It includes natural forces and human activities. All these causes are the results of activities that can increase the shear stresses or decrease shear strength of the soil mass. According to the Highway Research Board (1978), the factors are:

a) Factors that causes increases in shear stresses in slopes:
   - Removal of support includes erosion by stream and rivers, glaciers and successive wetting and drying; natural slope movement for example falls, slides

and settlements; and human activities such as excavations and removal of retaining walls and sheet pile.

- Overloading by natural causes such as weight precipitation (e.g heavy rainfall, snow) and by human activities such as construction of fill, buildings and other overloads at crest and water leakage (in culverts, water pipes, etc.)
- Transitory effects such as earthquake. Earthquake induces dynamic forces that reduce shear strength and stiffness of the soil. Pore water pressure in saturated coarsed-grained soil could rise to a value equal to a total mean stress and cause these soils to behave like viscous fluids (dynamic liquefaction). Structures founded on these soils will collapse; structures buried within them would rise. The quickness in which the dynamic forces are induces prevents even coarse-grained soils from draining the excess pore water pressure. Thus, failure occurs under undrained conditions.
- Removal of underlying materials that provided support by rivers, weathering, underground erosion due to seepage and human activities such as mining.
- Increase in lateral pressure such as by water in cracks and fissures and by expansion of clays.

b) Factors that causes decreases in shear strength in slopes:

- Factors inherent in the nature of the materials including composition, structure, and stratification
- Changes caused by weathering and physiochemical activity
- Effect of pore pressures
- Changes in structure such as stress release and structural degradation

2.2.4 Existing Control of Slope Failure and Problems Associates with the Control

Existing Control of Slope Failure

- Revegetation

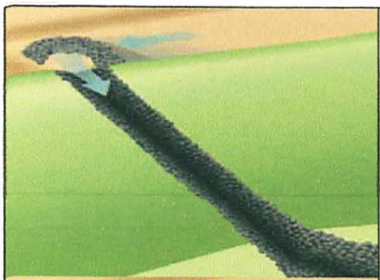
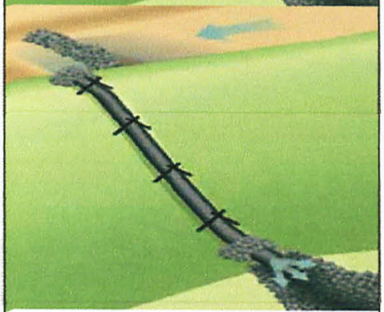
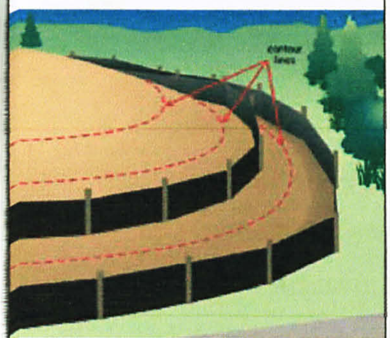
Erosion is the natural process where by external agent such as wind can remove soil particles. In the wet tropics involve rainfall; responsible for removal of surficial layers, resulting in rills and gullied deepen, causing slope to overstep. It will induce instability. Nature has designed vegetation as means to blanket and stabilize the good earth. It has evolved into rainforests, which comprise canopy of trees. These cover the organic human-rich topsoil, and offer excellent overall leaf litter layers (Hengchaovanich, 2004). Turfing is carried out using a broad-leaf carpet grass (*Axonopus compressus*) is available shade-tolerant grass that thrive well in residual soils with high rainfall. This is considered as the best method as it can gives instant covers and protection.

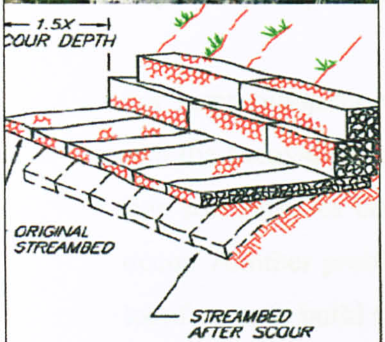
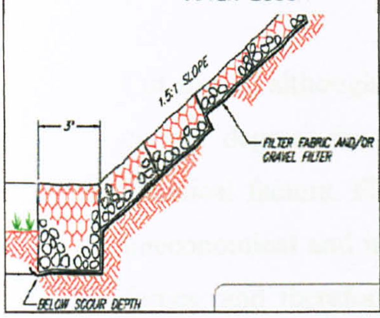
- Engineering Control

There are a lot of engineering controls for slope has been developed. Among the engineering controls are hand-dug-caisson, pump-and-treat system, horizontal drain, soil nailing, micropile, slope drains; gabion structures; and cellular mats. Table 2.2 shows some of the existing slope controls and the design criteria of the controls.

Table 2.2 Existing Slope Controls and Design Criteria

Slope Protection	Definition & Purpose	Design Criteria
Temporary Slope Drain 	A temporary slope drain is a pipe or lined (turf reinforcement mat, rock, or concrete) ditch or channel extending from the top to the bottom of a cut or fill slope during the construction period. Purpose: To convey concentrated runoff down the face of a cut or fill slope without causing erosion. Generally used in conjunction with diversions to convey runoff down a slope until permanent water management measures can be installed	***General:*** It is very important that these temporary structures be sized, installed, and maintained properly, because their failure will usually result in severe erosion of the slope. The entrance section to the drain should be well-entrenched, staked down, and stable so that surface water can enter freely. The drain should extend downslope beyond the toe of the slope to a stable area or appropriately stabilized outlet. ***Pipe Capacity:*** Peak flow from the 10-year, 24-hour storm. Multiple pipes or channels are often required for large areas, spaced as needed. ***Conduit:*** Construct slope drain pipes from heavy-duty, flexible materials

 		such as nonperforated, corrugated plastic pipe, or open top overside drains with tapered inlets, or CMP. ***Entrance:*** Construct the entrance to the slope drain of a standard flared-inlet section of pipe with a minimum 6-inch metal toe plate. Make all fittings watertight. ***Temporary Diversion:*** Use an earthen diversion with a dike ridge or berm to direct surface runoff into the temporary slope drain. ***Outlet Protection:*** Protect the outlet of the slope drain from erosion with an energy dissipator.
Silt fence  	A silt fence is a temporary sediment barrier consisting of filter fabric entrenched into the soil and attached to supporting posts. Silt fences are located downhill from bare soil areas and are installed with a trencher or by a slicing machine to prevent against common silt fence failures	Silt fencing must only be installed where water can pond. Specify silt fencing downgradient from bare soil areas, installed on the contour if possible, with the ends turned up to prevent bypassing. Provide adequate setbacks from slope toe for routine maintenance and access. Silt fencing can be used where: • Non-concentrated sheet flow will occur • Protection of adjacent property or nearby surface waters is required • The size of the drainage area is no more than 1/4 acre per 100 linear feet of silt fence • Small swales with bottoms 4 ft or wider are carrying sediment, the slope is less than 2 %, the drainage area is less than 2 acres and fences are installed in a series 50 feet apart in the swale.

Gabion Basket & Mattress    	Gabions are rectangular galvanized wire baskets filled with stones used as pervious, semiflexible building blocks for slope and channel stabilization. Live rooting branches may be placed between the rock-filled baskets. <u>Purpose:</u> Gabions protect slopes and streambanks from the erosive forces of moving water. Rockfilled gabion baskets or mattresses can be used as retaining walls for slopes, to armor the bed and/or banks of channels, or to divert flow away from eroding channel sections. Rock-filled or vegetated rock gabions are used on streambank sections subject to excessive erosion due to increased flows or disturbance during construction. Gabions can be specified where flow velocities exceed 6 ft/sec and where vegetative streambank protection alone is not sufficient.	***Gabion Wall:*** A gravity wall that relies on its own weight and frictional resistance to resist sliding and overturning from lateral earth pressure. ***Vegetated Rock Gabion:*** A rock-filled gabion earth-retaining structure which has live branches placed between each consecutive layer of rock-filled baskets. ***Gabion Deflector:*** Deflector or groins project into the streams and divert flows away from eroding streambank sections. ***Gabion Aprons:*** Rock filled gabions or gabion mattress used as outlet protection, energy dissipators or spillways. ***Grade Control:*** Drop structures or weirs. Gabion baskets and mattresses can be combined to construct check dams or weirs.

Another slope protection is sump pumping. This system is such a dewatering method. It involves allowing groundwater to seep into the excavation, collecting it in sumps and then pumping it away for disposal. This method can be very effective and economic method to achieve modest drawdowns in well-graded coarse soils or in hard fissured rock (Cashmen & Preene, 2001).

Cut slope is also one of the control methods. Shallow and deep cuts design is to determine a height and inclination that is economical and will remain stable for a reasonable life span. The design is influenced by the purpose of the cut, geological conditions, in situ material properties, seepage pressures, construction methods and potential occurrence phenomena such as heavy precipitation, flooding, etc. (Abramson, et. Al, 2002)

Problems Associates with the Existing Controls

Some of the existing control cannot give full protection or is not very effective. For example the turfing method, when due to various factors such as heavy demand, lack of good nurseries, and labor shortage, it has been overtaken by hydroseeding on project required mass production. Hydroseeding is suited for slope with little or no access.

For sump pumping system, sometimes it can lead to major problem. The flow of water into the excavation can have a destabilizing effect on fine-grained soils. This can lead to fine soil particles being washed from the soil with the water and the slope failure may occur. Another problem associates in existing control is the cost. Some controls need a lot of costs to build the system and to maintain it.

Cut slopes, although stable in short term, can fail many years later without warning. To a certain degree, the steepness of a cut slope is a matter of judgment not related to technical factors. Flat cut slopes, which may be stable for the indefinite period, often uneconomical and impractical. Long term cut slope stability also dependent on seepage forces, and therefore, on the ultimate groundwater level in slope (Abramson, et. Al, 2002).

Gabion structures are not recommended for steeply sloping channels where rock or high volumes of gravel sediment move at high velocity in the channel bed due to the possibility of damage to the wire mesh and failure of the basket or mattress structure.

Reasons for the high failure rate of improperly designed (located) and installed silt fence include the improper placement on the site, allowing excessive drainage area to the silt fence structure, mustow trenches with little or no soil compaction, inadequate attachment to posts, and failure to maintain the silt fence after installation

2.2.5 Fundamental of Control for Slope Failure

- *Reduce the slope instability.* The stability of slope can be increase by slope protection; agriculture and engineering methods.
- *Reduce the pore water pressure.* The shear strength of the soil mass will depend on the degree of suction negative pore pressure in the pore water pressure profile. The negative pore water pressure is to increase the shear strength. The influence of suction should be included by increasing the total cohesion according to the measured values of matric suction within the slope.
- *Reduce the infiltration level can control the slope failure.* Shallow landslide failure often be investigated using infinite slope analysis. If each slide of infinitely long slope is subject to the same amount and intensity of rainfall, an individual slice can be treated as a one-dimensional soil column subject to vertical infiltration. Infiltrations confirms that the resulting pore pressure profiles are identical for an infinite slope (Collins & Znidarcia, 2004).
- *Lower the groundwater level.* Local lowering of the water table or groundwater level and interception of any seepages due to perched water tables which might otherwise emerge on the exposed slopes or base of excavation. So the installed and operated of lowering groundwater ensures that construction work can be executed safely and economically

2.2.6 Groundwater effects on Slope Stability

Groundwater is one of the major factors in slope stability analysis. The flow of groundwater is usually very slow and laminar flow. The groundwater near the wall of interstices is held motionless by molecular attraction of the wall. The energy that causes groundwater to flow is derived from gravity. The relationship regarding the rate of movement was developed by Henri Darcy in 1856; it states that

$$Velocity,\ v = k\,(h/l) \qquad (Eq.\ 2\text{-}1)$$

where h = the head

l = the length flow

k= a coefficient that depends on the permeability of the material, the acceleration of gravity, and the viscosity of water

Data for pore pressure calculation on the anticipated critical failure profile can be done by constructing the phreatic surface for an unconfined and piezometric line for confined aquifer. Figure 2.8 shows a comparison between phreatic and piezometric pore pressure calculation.

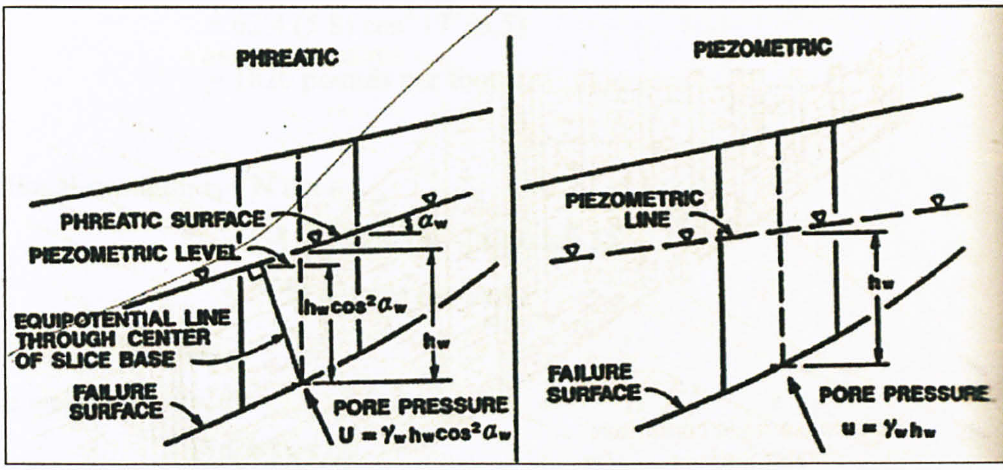


Figure 2.8 Comparison between Phreatic and Piezometric Pore Pressure Calculation (Abramson, et. Al, 2002)

Groundwater can affects the slope stability by three ways; reduction in shear strength, reduction in frictional strength, and effects of seepage direction (Abramson, 2002).

Reduction in shear strength. Saturation of soil will decrease the frictional shear strength. It is due to the buoyant reduction in normal force required for frictional shear strength by the pore pressure. Saturation of soil may also destroy capillarity and apparent cohesion on the cohesive component of soil, or may be reduce the dry strength of a cohesive soil.

Reduction in Frictional Strength. It is obvious that frictional strength is higher without groundwater. As shown in Figure 2.9, this reduction can be approved in the calculation below:

With groundwater;

Slice weight, $W = [\gamma_t (d - d_w) + \gamma_{sat} d_w]$

$b = [120(8.3 - 5.8) + 135(5.8)]5$

$W = 5415$ pounds per foot

Pore water force, $U = \gamma_w d_w \cos^2 \alpha_w L$

$= 62.4 (5.8) \cos^2 17^0 (5.5)$

$= 1820$ pounds per foot

Frictional strength, $\tau_f = N \tan \theta$

$= (5415 \times \cos 26^0 - 1820) \tan 35^0$

$= 2134$ pounds per foot

Without groundwater;

Slice weight, $W = \gamma_t db$

$= 120(8.3)(5.0)$

$W = 4980$ pounds per foot

Pore water force, $U = 0$ since $d_w = 0$

Frictional strength, $\tau_f = N \tan \theta$

$= (4980 \times \cos 26^0 - 0) \tan 35^0$

$= 3134$ pounds per foot

Groundwater reduces frictional strength by;

$[(3134 - 2134) / 3134] \times 100\% = 32\ \%$

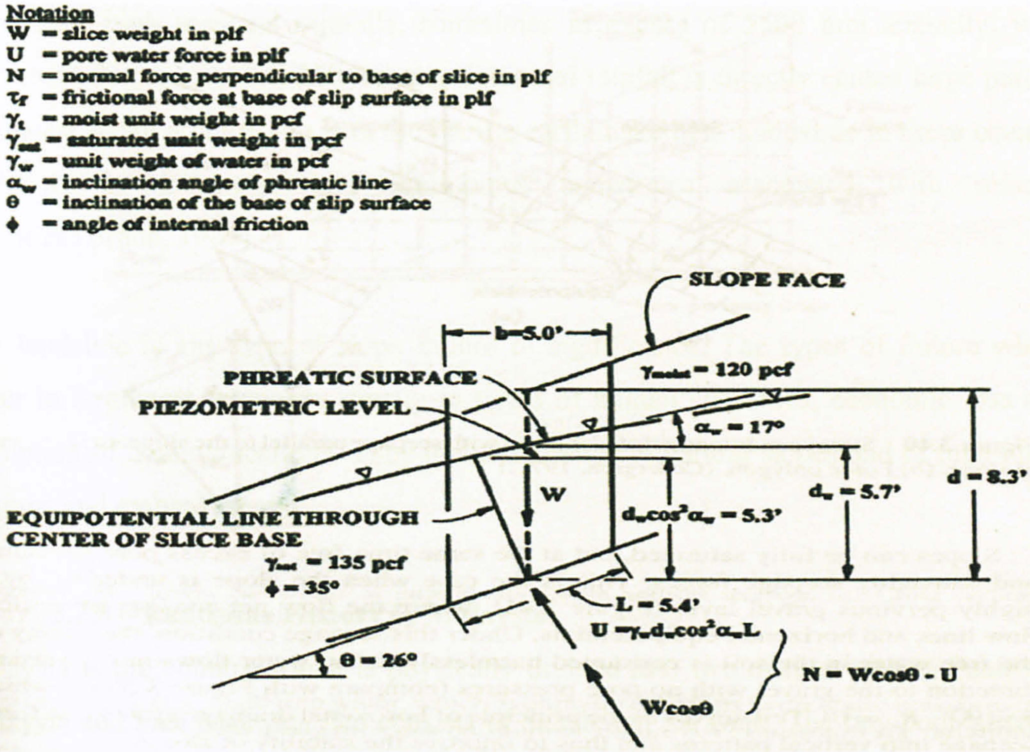


Figure 2.9 Reduction in Frictional Shear Strength due to Groundwater (Prellwitz, 1990)

Effect of seepage direction. Many slope become saturated during period of intense rainfall, with the water table rising to the ground surface, and water flowing essentially parallel to the direction of the slope. Using the hydraulic gradient method, the seepage force F can be determined from the flow net. This seepage force F acts a driving force in the soil mass hence can greatly lower the stability of the slope.

2.2.7 Landslide History in Southeast Asia

There are so many cases of landslide happened in Malaysia. This is because the slope failure and the ineffective of slope controls. The Southeast Asian region is physiographically and geologically as complex as any area of the world (Bemmelen, 1949). With notable exceptions of some large deltaic plains, the majority of terrain of the total land area of Southeast Asia is hilly and mountainous. The mainly warm, wet climatic conditions have resulted in varying depths of weathering of a wide range of igneous metamorphic and indurated sedimentary rocks, to give profiles which grade from residual soils at the surface through to unweathered bedrock at depth. The region experience high seasonal rainfalls, sometimes in excess of 5500 mm annually, with intensities that can exceed 150 mm per hour, and rainfall is directly causes large part of Southeast Asian archipelago is in the Pacific earthquake belt. Landslide in these country such as in Indonesia and Philippines sometimes associated with seismic activities.(Brand, 1989)

The landslide is any type of slope failure of significance. The types of failure which occur in Southeast Asia vary greatly in terms of human casualties, economic loss and disruption of communications. They include failures of natural slopes and of man-made cuttings and embankments.

2.2.8 Landslide History in Malaysia

Malaysia is the country which is physically divided into two distinct pieces; Peninsula Malaysia and East Malaysia that consists of Sabah and Sarawak. The tiny population of East Malaysia is found almost entirely along the relatively flat land of the north coast, and there are a few building developments or roads into the hills to the south. There are only one mention of landslides problems near a centre of population in Sabah (Hunt, 1971). The topography of West Malaysia (Figure 2.10) is a natural continuation of that along peninsula Thailand.

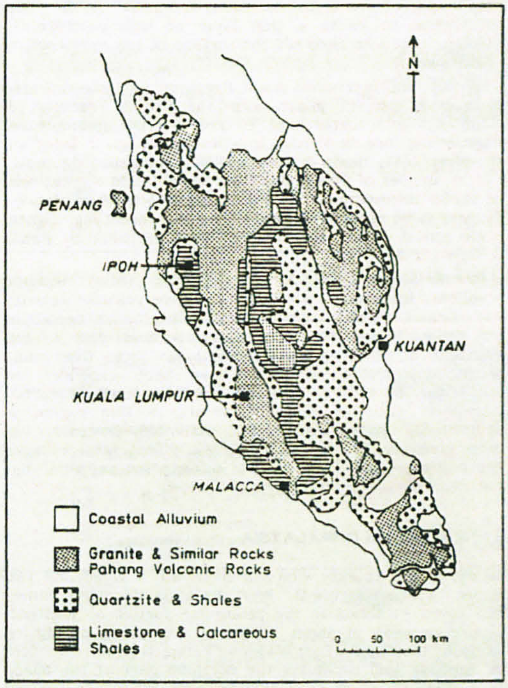


Figure 2.10 Geological Map of West Malaysia (Brand, 1989)

The igneous rock, the majority of which are granites, weather to form sandy clay soils, which often contain fine gravel size quartz particles, overlying grade to I to III rocks. The sedimentary rocks range from firm quartzite and sandstone to soft phyllites and shales, both give a Grade IV rocks. The peninsula is hot and wet all the year round, with an average annual rainfall of about 3100 mm on the east coast, decreasing to about 2500 mm on the west coast. Rainfall intensities can exceptionally reach more than 100 mm/hour in some places (Ministry of Agriculture, 1977).

In the mountainous regions of West Malaysia, landslides in natural slopes are fairly common (Ting, 1984). These usually take the form of shallow slide, 3 to 4 m deep, in the residual soils mantle to give failure surfaces almost parallel to the slope face. The rapid economic development in Malaysia over last four decades has resulted in the construction of many new roads and buildings. The variations in the residual materials from the geological and the weathering points of view are such that it is not generally possible to apply satisfactory slope design procedures, engineering judgement and precedent being replied upon for the determination of cut slope angles.

The Kuala Lumpur-Karak Highway, running west to east across the mountains, has been particularly badly affected by landslides (Tan, 1987; Moh et al, 1987). The East-West Highway across the mountains in the north of the country has also suffered badly. Failure in cut slopes are very often initiated by the occurrence of surface erosion, which sometimes takes place rapidly immediate after a cut slope is formed (Tan & Ting, 1984). A serious example shown in Figure 2.11.

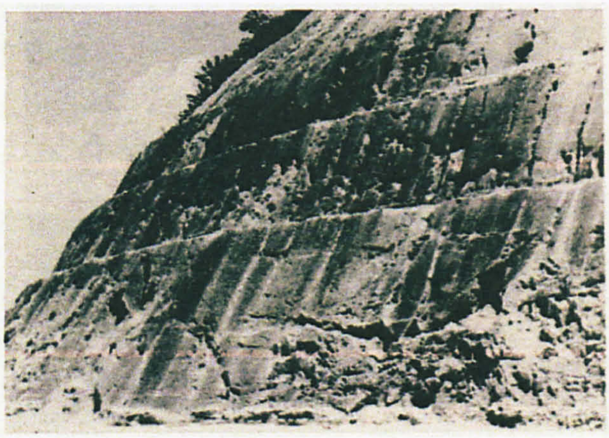


Figure 2.11 Surface Erosion in 30 m High Granite Cut Slope (Brand, 1989)

The most serious landslide in Malaysia have been of unusual cause, in that they have been associated with the open excavations used to pursue tin mining in the northwestern part of the country. Because of its bad record of casualties, the tin mining in Malaysia has recently begun to replace its previous ad hoc methods of slope construction by properly engineered techniques which include the use of stability analysis (Brand, 1989).

There are a lot of causes from the recent investigation of slope failures in Malaysia. The causes are heavy rains, deforestation and improper construction of piling. Some of the landslides histories in Malaysia are shows in Table 2.3. Most of the cases found in granite area. Granite is the most critical soil that lead to slope failure (Brand, 1989). It provides good foundation for building construction but soil originated from granite contain substantial amount of clay that may decrease the strength of the soil.

Table 2.3 Landslides Histories in Malaysia

Year	Province (State)	Triggering Process	Impact	Comments
1993	Highland Towers (Kuala Lumpur)	Heavy rains	48 killed, 12-storey condominium collapse	
June, 1995	Karak Highway, (Pahang)		20 killed	Tons of land fall down on highway
August, 1996	Pos Dipang, Kampar (Perak)	Mud flow	44 killed	
May, 1999	Bukit Antarabangsa, Ulu Kelang (Selangor)	Heavy rains	More than 10000 people trapped	50000 m2 land around the area fall down hill onto 100 m road nearby
November, 2002	Taman Hillview, Ulu Kelang (Selangor)	Heavy rains	8 killed	Collapse of retaining wall causes the landslide, heavy rains cause foundation of the house or clay soil underneath it being saturated with water
October, 2006	Wangsa Maju (KL)	Soil erosion	700 people trapped	The highway construction, burst underground pipe have caused soil erosion on the hill
March, 2007	Putrajaya	Heavy rains	Evacuation of 1000 residents, buried 23 cars	Tons of earth tumbled down a hill 10 meters behind an apartment

CHAPTER 3

METHODOLOGY

This study carried out by some methodology started with soil sampling, laboratory testing, design and modelling the trenching and finally analyse the factor of safety for the slope using Slope W software.

3.1 Project Activities

3.1.1 Soil Sampling, Laboratory Testing and Analysis of Laboratory Data

The author went to Bukit Kledang, Perak to take the samples of granite soil and then brought them to the lab. The samples were taken at the middle part of the hill so that the soil is taken from the suitable place according to the soil profile.

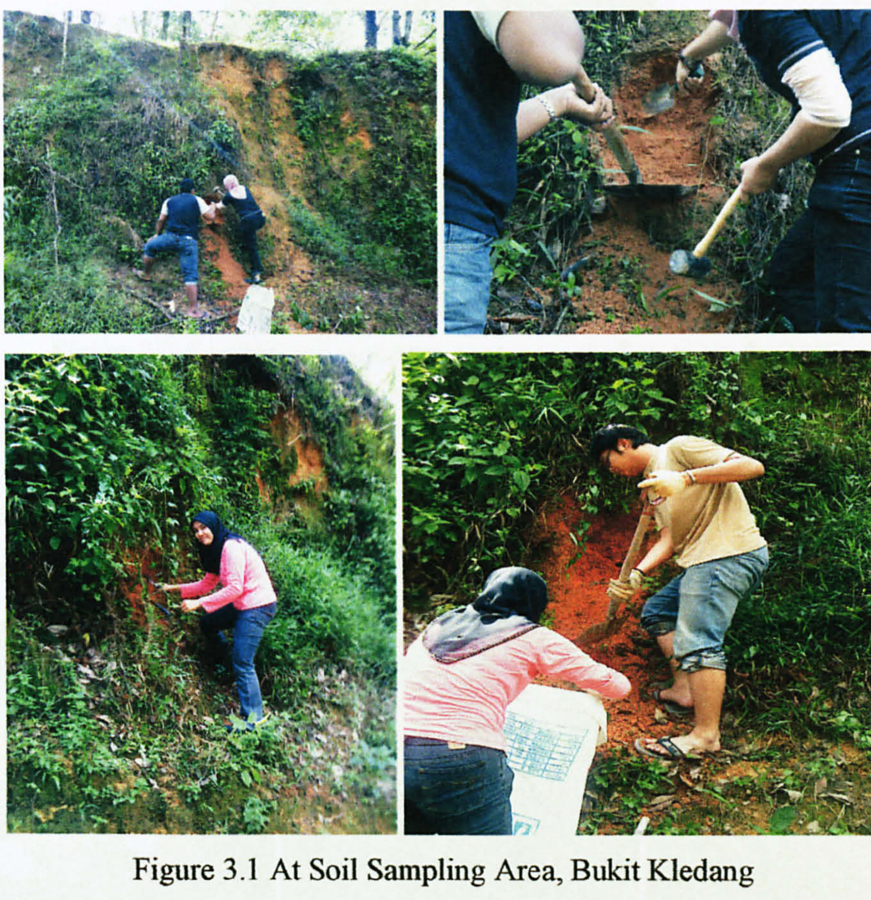


Figure 3.1 At Soil Sampling Area, Bukit Kledang

After taking the soil samples from preferred location, the samples was taken to the Geotechnical Laboratory for testing on some engineering properties of the soil such as moisture content, shear strength, hydraulic conductivity, specific gravity, particle size distribution, liquid and plastic limits. This testing was conducted after perfecting the technique and analysis according to certain standard for those properties. The tests that were conducted include oven drying method, pH test, specific gravity test, particle size distribution test, plastic limit test, cone penetrometer test, falling head and constant head tests, hydrometer test, and direct shear box test. *(Refer to Appendices for laboratory procedure of each test).*

a) Oven Drying Method

Objective: To determine the moisture content in soil.

Apparatus: Drying oven, moisture content tins and electronic balance

Figure 3.2 Samples of moisture content test after oven-dry test

b) pH Test

Objective: To determine the pH value of the soil.

Apparatus: pH meter, beaker, distilled water.

Figure 3.3 Samples tested by pH Test

c) Particle Density or Specific Gravity Test

Objective: To determine the value of particle density or specific gravity, ρ_s of soils using the pyknometer method

Apparatus: A set of pyknometer, glass rod, electronic balance, thermometer and drying oven

(a) (b)

Figure 3.4 The pyknometer test (a) before and (b) after 24 hours

d) Particle Size Distribution Test

Objective: To determine the size distribution of soil using the dry sieving

Apparatus: Test siever from size 2 mm until 63 μm, lid and pan, electronic balance, drying oven, tray, scoop, sieve brush and mechanical sieve shaker

Figure 3.5 Test Sieves on the Mechanical Shaker

e) Determination of Plastic Limit Test

Objective: To determine the plastic limit and plasticity index of soil

Apparatus: A flat glass plate, spatulas, and rod compactor

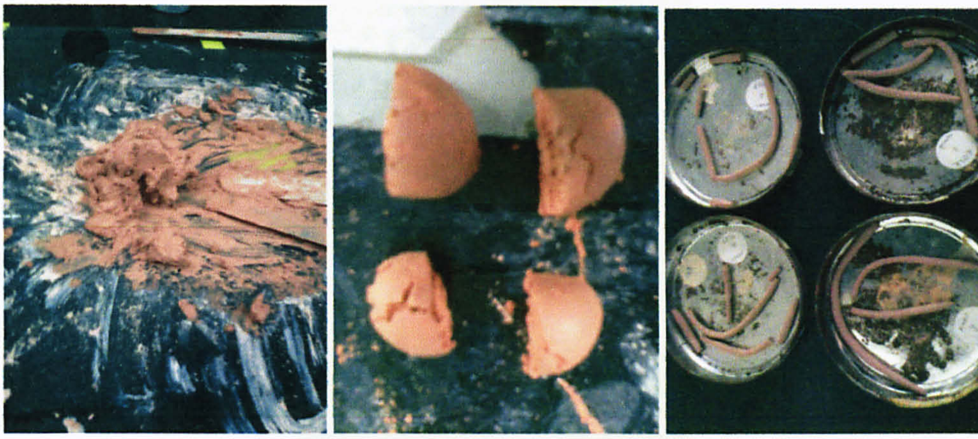


Figure 3.6 Procedure of Plastic Limit Test

f) Cone Penetrometer Test

Objectives: To determine the liquid limit of soil using cone penetrometer

Apparatus: A flat glass plate, spatulas, straightedge, cone penetrometer, metal cup, evaporating dish, wash bottle, automatic controller which release the plunger head

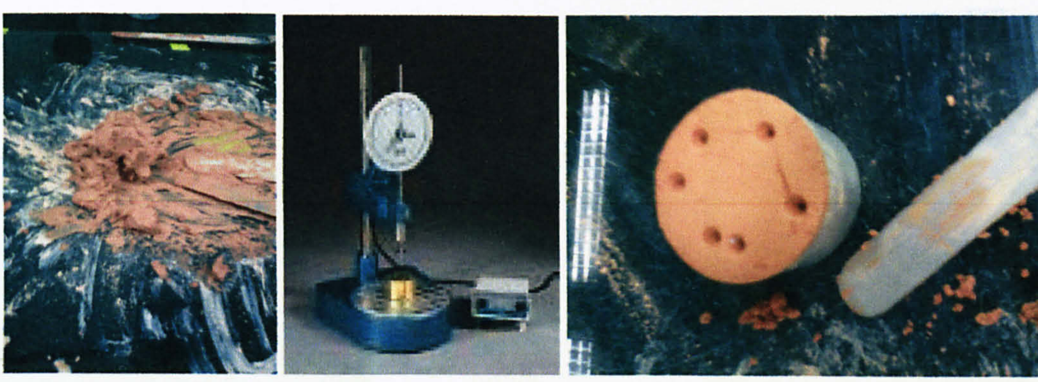


Figure 3.7 Procedure of Liquid Limit Test

g) Falling Head Test

Objective: To determine the hydraulic conductivity of sand sample by using Falling Head Permeability Method

Apparatus: Permeameater cell, manometer tube of glass plastic, filter material, measuring cylinder, stop watch readable

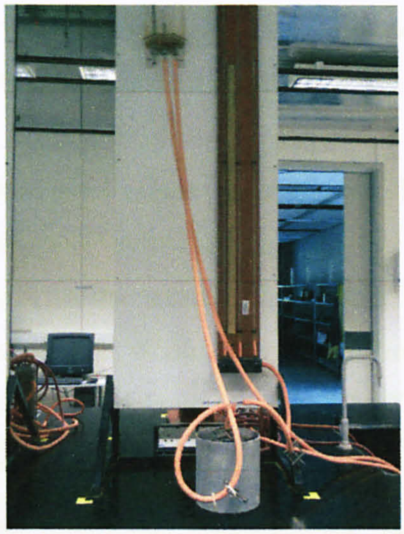
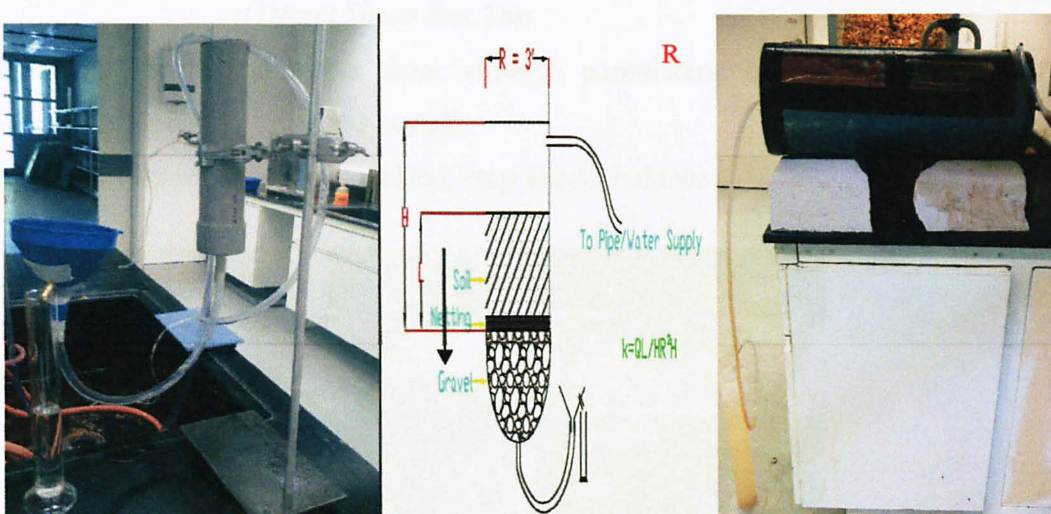


Figure 3.8 Falling Head Permeability Cell

h) Constant Head Test

Objective: To determine the hydraulic conductivity of sand sample by using Constant Head Permeability Method

Apparatus: 3" pipe with end cap, Hose, Funnel, Measuring cylinder, Stop watch



(a) (b)

Figure 3.9 (a) Constant Head Permeability Cell and the Cross Section (b) Constant Head for Soil with Modification (Addition of CaOH) *i) Sedimentation by the Hydrometer Method*

Objective: To determine the particle size distribution by the hydrometer method as according to BS 1377 Part 2. This method covers the quantitative determination of the particle distribution in a soil from the coarse sand size to the clay size.

Apparatus: Hydrometer, graduated glass measuring cylinders with ground glass stoppers, thermometer, test sieves, balance, drying oven, desiccators, evaporating dishes, wide mouth of clonical flask, measuring cylinder, constant temperature bath, Sodium hexametaphosphate solution, dissolve 40 g of the solution in distilled water to make 1 L of solution

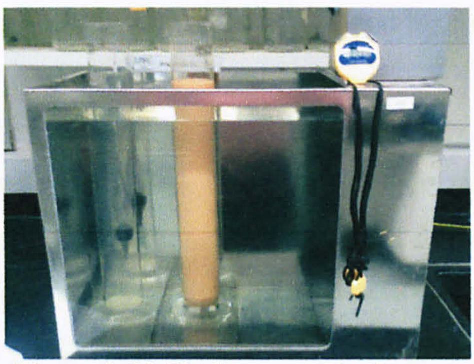


Figure 3.10 Hydrometer Equipment

j) Direct Shear Box Test

Objective: To determine the shear strength parameters, c’ and angle of shearing resistance, □’ of the sample.

Apparatus: Direct shear box machine, stop watch, balance.

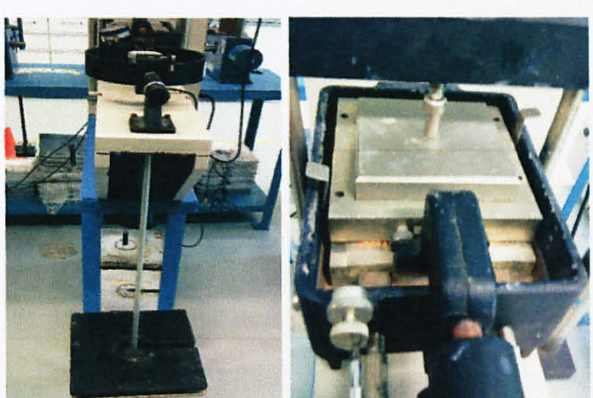


Figure 3.11 Direct Shear Box Machine

3.1.2 Design and Modeling the Trenching

In this stage, the design of the trenching is implemented according to the type of soil and the effectiveness of the design. After finish and satisfied with the design, the model for the design is developed. Then, the moisture level of the soil determined to see the effectiveness of the model.

3.1.3 Analyse the Factor of Safety by Slope/W Software

The software was used during the analysis stage, after the completion of the design and modeling process. It is a slope stability software for computing the factor of safety of earth and rock slopes that can analyze problems of slip surface shapes, pore-water pressure conditions, soil properties, analysis methods and loading conditions.

3.2 Tools and Equipment

There are some tools need in order to carry out the soil testing and implement the project. This is to ensure that the project can be accomplished within the given timeframe. All the tools and equipment used for laboratory tests are stated above. The software that used during this project is Slope/W

3.3 Project Chronology

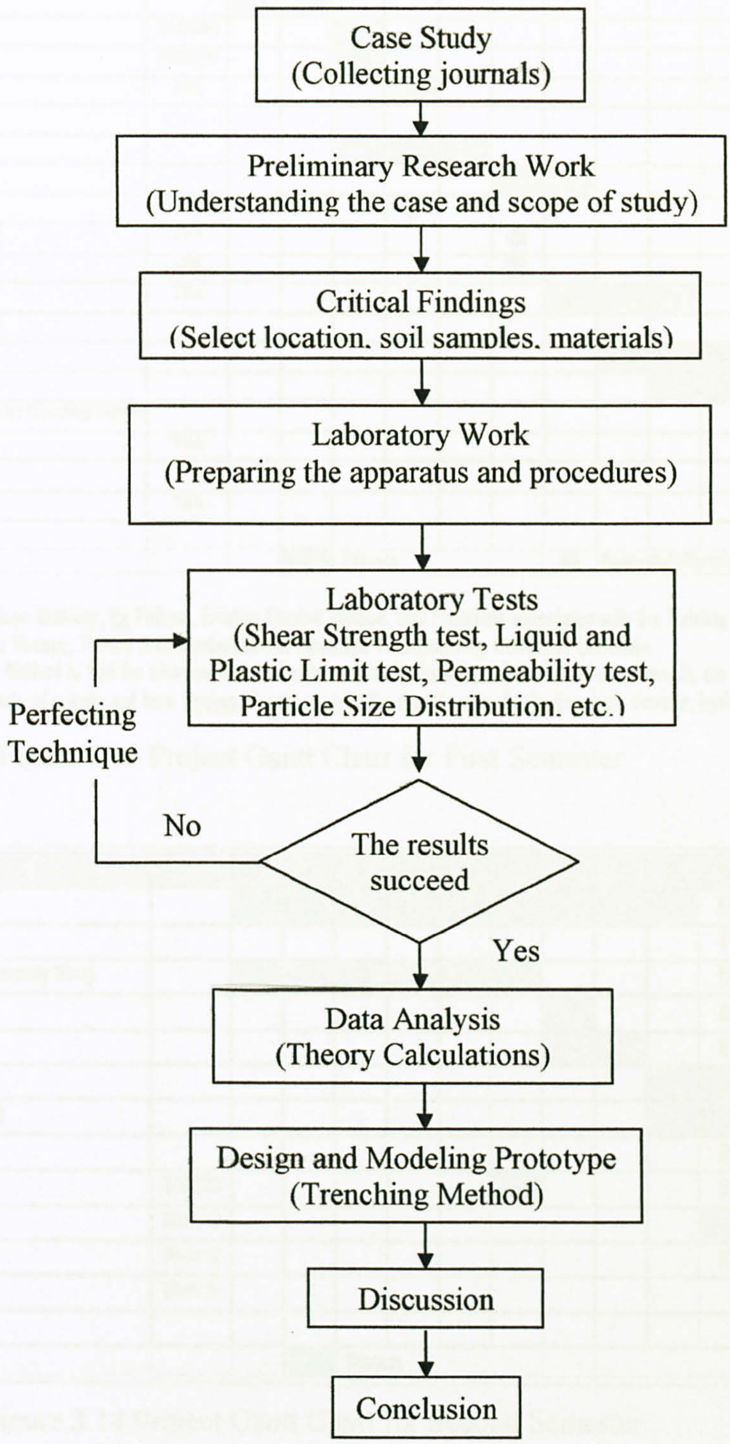


Figure 3.12 Project Chronology

3.4 Gantt Chart

No.	DETAIL/WEEK	DATE	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1	Selection of Project Topic		■	■													
2	Briefing (Library Session)	04/08/08			■												
3	Seminar 1 (by Prof Celia)	06/08/08			■								M				
4	Briefing by LFSU	TBA				■							I				
5	Research Work												D				
	5.1 Literature Review 1				■	■	■ 1										
	5.2 Literature Review 2							■	■ 2				S				
6	Submission of Journal 1	11/08/08				●							E				
7	Submission of Preliminary Report	TBA						●					M				
8	Submission of Progress Report	TBA						●									
9	Seminar 2	TBA							■	■	■		B				
10	Project Work												R				
	10.1 Perfecting Technique									■	■	■ 3	E				
	10.2 Soil Sampling & Analysis										■	■	A	■ 4			
	10.3 Preliminary Design of Model for Trenching Method												K	■	■		
11	Submission of Interim Report	TBA												●			
12	Project Work Continues														■	■	
13	Oral Presentation	TBA															●
				■ Process					● Suggested Milestone								

1 Literature Review on Slope; Slope Stability, it's Failure, Existing Control Method, and Problems associates with the Existing Control
2 Literature Review on Landslide History, Theory & Characteristics of Landslide & Engineering Control of Landslide
3 Perfecting Technique/ Testing Method to find the characteristics of soils such as liquid limit, plastic limit, shear strength, etc
4 Soil Sampling & Analysis (sample of granite soil from Pusing, Perak, analyse the particle size distribution, bulk density, hydraulic conductivity, etc)

Figure 3.13 Project Gantt Chart for First Semester

No.	DETAIL / WEEK	DATE	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1	Research Work		■	■	■	■	■	■	■	■	■	M					
2	Project Work											I					
	2.1 Perfecting Technique (Laboratory Work)		■	■	■	■	■	■				D					
	2.2 Design of Trenching								■			S					
	2.3 Modelling the Trench								■	■		E					
	2.4 Model Testing										■	■ M					
	2.5 Analyse Data (by Software)										■	■ B	■	■			
												R					
3	Progress Report Submission	26/02/09						■				E					
4	Poster Presentation	Week 10										■ A					
5	Dissertation (First Draft)	Week 12										K		■			
6	Oral Presentation	Week 15															■
				■ Process													

Figure 3.14 Project Gantt Chart for Second Semester

CHAPTER 4
RESULTS AND DISCUSSION

After the laboratory tests were conducted, all the results were tabulated according to the test. The author has done the test twice to get the best results. All the calculations were calculated using some formula stated in each test result and some of the soil properties gathered.

4.1 Oven-Drying Method (Moisture Content)

The amount of water, expressed as a proportion by mass of the dry solid particles which is the moisture content has a profound effect on soil behaviour. The soil is dry when no further water can be removed at temperature not exceeding 110^0 C. Moisture content is required as a guide to classify the natural soils and as a control criterion in re-compacted soils and is measured on samples used for most field and laboratory test. So below is the result of the test.

Table 4.1 Results of Moisture Content Test

Container No.	1	2	3
Mass of wet soil + container, m_2 (g)	86.94	84.64	88.37
Mass of dry soil + container, m_3 (g)	76.91	74.96	78.04
Mass of container, m_1 (g)	19.93	19.75	21.1
Mass of moisture, $m_2 - m_3$ (g)	10.03	9.68	10.33
Mass of dry soil, $m_3 - m_1$ (g)	56.98	55.21	56.94
Moisture content, $W = \frac{m_2 - m_3}{m_3 - m_1} \times 100\%$	17.6%	17.5%	18.14%

From the test, the author found that the average moisture content of the soil is;

$$\text{Average moisture content} = \frac{(17.6 + 17.5 + 18.14)\%}{3} = 17.75\ \%$$

It is normal moisture content for granite soil but if the weather is not really good such as in rainy day or else, the moisture content will be higher. When dealing with the design later, this value of moisture content need to be considered because the slope failure will happen if the moisture content is higher than this value.

Besides moisture content, the author conducted test to samples that was modified using 5% of Calcium Hydroxide (CaOH). The results of the moisture content analysis after running the model of modified sample are shown in Table 4.2 below.

Table 4.2 Results of Moisture Content Test (with modification)

Container No.	1	2	3
Mass of wet soil + container, m_2 (g)	77.96	86.99	85.13
Mass of dry soil + container, m_3 (g)	68.86	77.02	75.48
Mass of container, m_1 (g)	18.60	20.68	20.51
Mass of moisture, $m_2 - m_3$ (g)	9.10	9.97	9.65
Mass of dry soil, $m_3 - m_1$ (g)	50.26	56.34	54.97
Moisture content, $W = \frac{m_2 - m_3}{m_3 - m_1} \times 100\%$	18.11%	17.69%	17.56%

From the test, the author found that the average moisture content of the soil is;

$$\text{Average moisture content} = \frac{(18.11 + 17.69 + 17.56)\%}{3} = 17.79\ \%$$

It is quite similar to the previous results. So the addition of Calcium Hydroxide is not really affecting the moisture content of the sample.

4.2 pH Test

Pure water has a pH around 7 which is neutral while when acid is dissolved, the pH will be less than 7 and when a base or alkali is dissolved in water, the pH will be greater than 7. The results of the test are as in Table 4.3:

Table 4.3 pH Result for Granite Soil

Sample No.	pH value	Acidic/Basic
1	4.79	Acidic
2	4.72	Acidic
3	4.61	Acidic
Average	4.71	Acidic

From the result, it shows that granite is an acidic soil. The value 4.71 is less than 7.

4.3 Particle Density or Specific Gravity Test

The specific gravity of soil solids often needed for various calculations in soil mechanics. The results are tabulated in Table 4.4.

Table 4.4 Results of Specific Gravity Test

Mass of jar + gas jar + plate + soil + water	m3	g	1811.50
Mass of jar + gas jar + plate + soil	m2	g	937.07
Mass of jar + gas jar + plate + water	m4	g	1562.15
Mass of jar + gas jar + plate	m1	g	537.07
Mass of soil	m2 – m1	g	400.00
Mass of water in full jar	m4 – m1	g	1025.08
Mass of water used	m3 – m2	g	874.43
Volume of soil particles	(m4 – m1)-(m3-m2)	ML	150.65
Particle density, ρ_s	$\frac{(m2 - m1)}{(m4 - m1)-(m3-m2)}$	Mg/m^3	2.655
Average value ρ_s		Mg/m^3	2.655

From this calculation, the specific gravity value for the soil is 2.655. This is the normal and suitable value for the soil because it is a light colored-sand, most of the sand should have the specific gravity between 2.6 to 2.7.

4.4 Particle Size Distribution Test

For this test, the author chose to do the dry sieve. Dry sieving is suitable only for soil containing insignificant quantities of silt and sand. The particle size distribution also can be carried out by hydrometer and pipette method because the soil is a fine soil. Combined sieving (dry and wet sieve) and sedimentation procedures enables a continuous particle size distribution curve of a soil to be plotted from the size of the coarsest particle down to the clay size. This method covers the quantitative determination of particle size distribution in a cohesionless soil down to the fine sand size. Table 4.5 shows the result of this test.

Table 4.5 Results of Particle Size Distribution

Initial Mass, m = 500.00 g					
Siever Size	Siever Mass (g)	Siever + Soil Mass Retained (g)	Mass Retained, m (g)	Percentage Retained (m/m_1) x 100%	Cummulative Percentage Passing (%)
2 mm	509.11	456.62	52.49	10.50	10.50
1.18 mm	491.86	425.74	66.12	13.22	23.72
600 µm	500.61	404.01	96.60	19.32	43.04
425 µm	435.87	378.80	57.07	11.41	54.45
300 µm	412.44	358.29	54.15	10.83	65.28
212 µm	391.53	346.14	45.39	9.08	74.36
150 µm	382.41	349.11	33.30	6.66	81.02
63 µm	353.59	321.27	32.32	6.46	87.48
Passing 63 µm	452.23	390.06	62.17	12.43	99.91
Total			**499.61**	**99.91**	

Once the percent finer for each sieve is calculated, the calculations are plotted in a semi-logarithmic graph paper (BS 1377: Part 2). This plot is referred to as the particle size distribution curve.

4.5 Sedimentation by the Hydrometer Method

The test was conducted to determine the particle size distribution that covers the quantitative determination of the particle distribution in a soil from the coarse to the clay size.

Details of calibration and sample data:

H = 69.53 mm h = 157.96 mm d_1 = 18.82 mm

N = 11.09 mm d_2 = 38.30 mm

Meniscus correction, C_m = 0.0005 mm d_3 = 57.87 mm

Volume hydrometer, V_h = 0.0067 kg d_4 = 77.52 mm

Reading dispersant, R_o' = 998 mm d_5 = 96.89 mm

Dry mass soil, m = 50 g d_6 = 116.45 mm

Particle density, ρ_s = 2.6 Mg/m^3 d_7 = 136.45 mm

Dynamic Viscosity, η = 0.891 mPa.s

Temperature, T = 25^{0}C

Length between 100 mL to 1000 mL = 312 mm

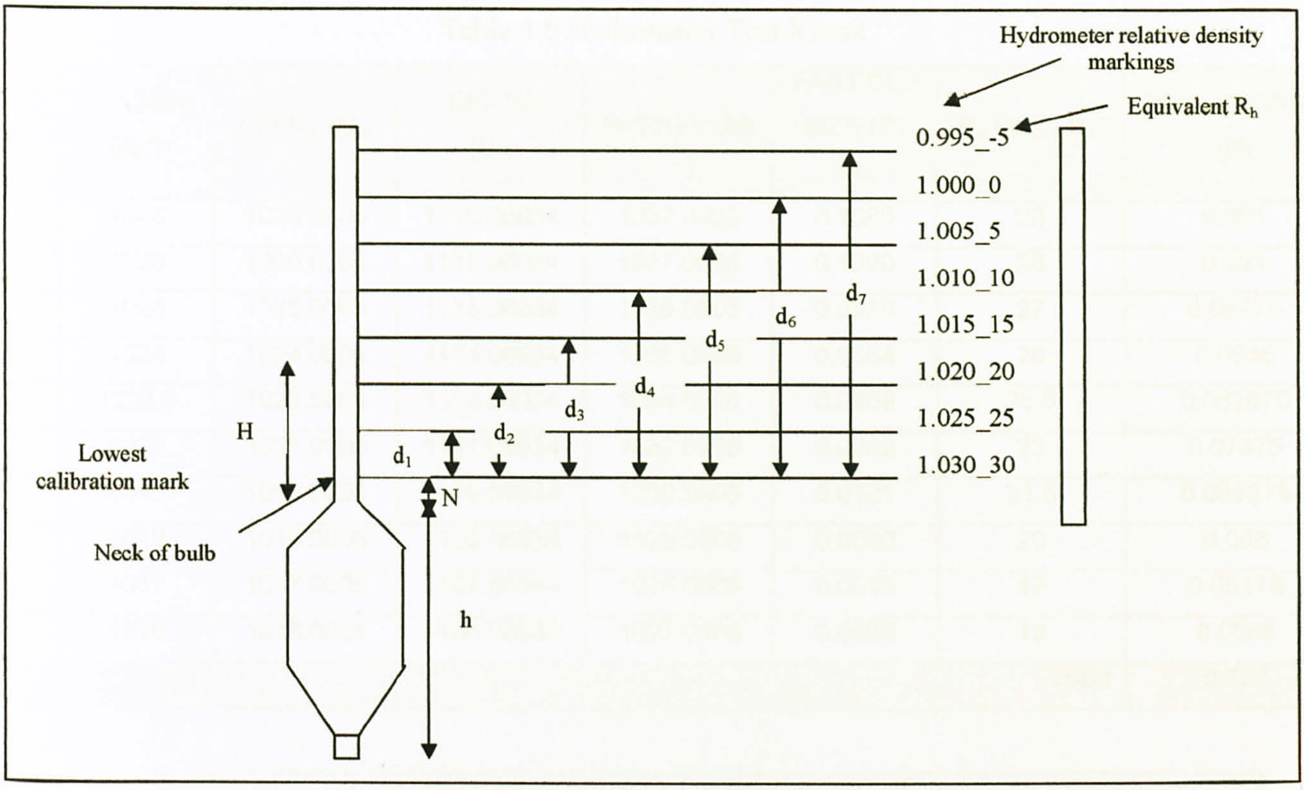


Figure 4.1 Schematic Diagram for Hydrometer

1. True hydrometer reading, R_h (mm) = $R_h' + C_m$

   Where R_h' = observed hydrometer reading

2. Calculate effective depth, $H_R = H + ½ [h - V_hL/900]$

   Where H_R is the length from the neck of the bulb to graduation Rh (mm)

   H is the length of the bulb

   V_h is the volume of hydrometer bulb (mL)

   L is the distance between the 100 mL and the 1000 mL scale markings of the sedimentation cylinder (mm)

3. Find the equivalent particle diameter, D (mm) = $0.005531 \sqrt{\eta}\, H/(\rho_s - 1)t$

4. Calculate the modified hydrometer reading, $R_d = R_h' - R_o'$

   Where R_o' is hydrometer reading at the upper rim of the meniscus in dispersion solution

5. Calculate the percentage by mass, K of particle smaller than D,

   $K = [100\, \rho_s / m\, (\rho_s - 1)] \times R_d$

Table 4.6 Hydrometer Test Result

ELAPSED TIME	READING (R_h')	$Rh=R_h'+C_m$	DEPTH (H_r)	H=Rh+11.09	PARTICLE SIZE (D) mm	$R_d=R_h'-R_o'$	PERCENTAGE (K)
0.5	1026	1026.0005	1116.06934	1037.0905	0.1880	28	0.091
1	1026	1026.0005	1116.06934	1037.0905	0.1329	28	0.091
2	1025	1025.0005	1115.06934	1036.0905	0.0939	27	0.08775
4	1024	1024.0005	1114.06934	1035.0905	0.0664	26	0.0845
8	1023.5	1023.5005	1113.56934	1034.5905	0.0469	25.5	0.082875
30	1021	1021.0005	1111.06934	1032.0905	0.0242	23	0.07475
120	1019.5	1019.5005	1109.56934	1030.5905	0.0121	21.5	0.069875
480	1018	1018.0005	1108.06934	1029.0905	0.0060	20	0.065
720	1017	1017.0005	1107.06934	1028.0905	0.0049	19	0.06175
1440	1016	1016.0005	1106.06934	1027.0905	0.0035	18	0.0585
						Total	**0.767**

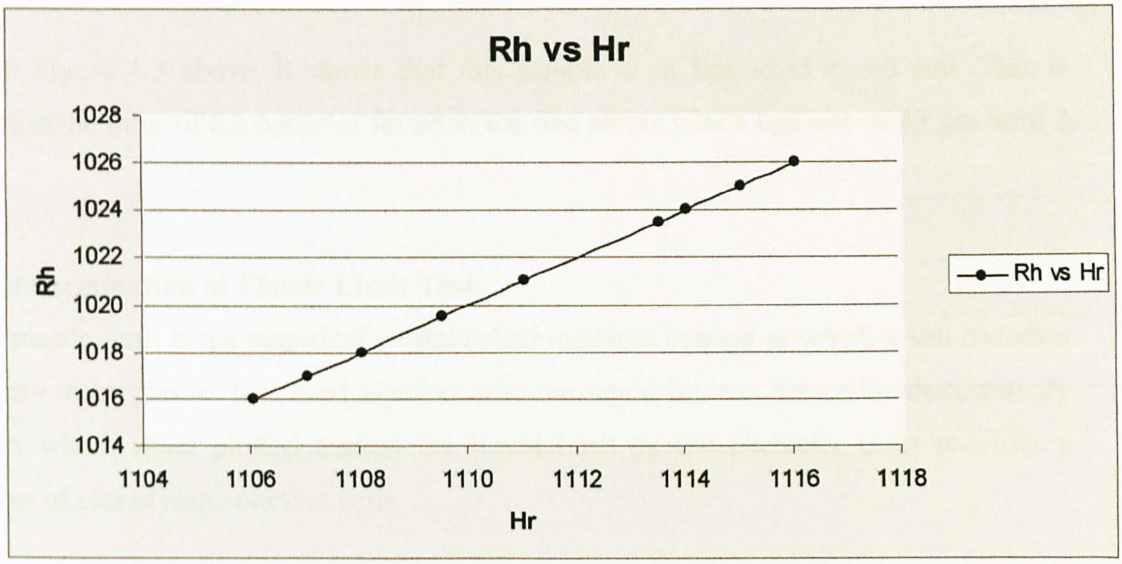


Figure 4.2 Graph of Rh vs Hr for Hydrometer Test

Table 4.6 shows the result of hydrometer test while Figure 4.2 shows the graph of Rh vs Hr for this test. From the graph, it shows that Rh is constant with Hr value.

By combining the sieve analysis and hydrometer result, the particle size distribution curve are plotted in a semi-logarithmic graph paper (BS 1377: Part 2)

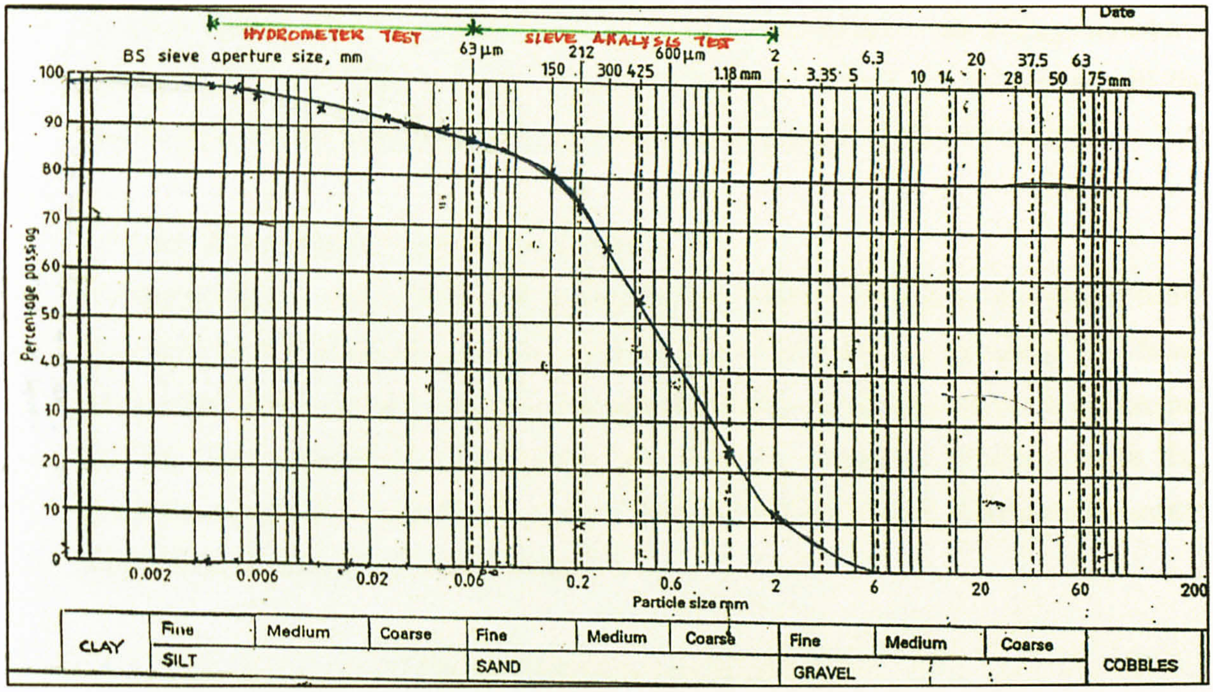


Figure 4.3 Particle Size Distribution Curve from Sieve and Hydrometer Test

From Figure 4.3 above, it shows that this sample is in fine sand to silt soil. This is because the most of the particles found in the fine siever which has size of 63 μm until 2 mm.

4.6 Determination of Plastic Limit Test

The plastic limit is the empirically established moisture content at which a soil becomes too dry to be plastic. It is used together with the liquid limit to determine the plasticity index which when plotted against the liquid limit on the plasticity chart provides a means of classifying cohesive soils.

Table 4.7 Results of Plastic Limit

Mass of wet soil + container (g)	24.59	27.63	24.44	27.05
Mass of dry soil + container (g)	23.41	25.74	23.11	25.17
Mass of container (g)	19.53	19.82	19.10	19.82
Mass of moisture (g)	1.18	1.89	1.33	1.78
Mass of dry soil (g)	3.88	5.92	4.01	5.45
Mass of wet soil + container (g)	30.41	31.93	33.16	32.66
Average moisture content	32.04 %			

Result of plastic limit test is tabulated in Table 4.7. From the table, the average moisture content is 32.04 %. The value is suitable for this type of soil and this value will be combined with the liquid limit value to determine the plasticity index for the soil.

4.7 Cone Penetrometer Test (Liquid Limit)

The liquid limit is empirically established moisture content at which a soil passes from the liquid state to the plastic state. Due to the difficulty in achieving the liquid limit from a single test, three or more tests was conducted at various moisture contents to determine the fall cone penetration. After getting the moisture contents, the author took the moisture content corresponding to a cone penetration of 20 mm to determine the exact liquid limit. Results are shows in Table 4.8.

Table 4.8 Results of Liquid Limit

Container No.	**1**			**2**			**3**		
Initial dial gauge (mm)	0	0	0	0	0	0	0	0	0
Final dial gauge (mm)	12.9	12.9	12.8	18.4	18.0	17.9	20.7	20.5	20.9
Average penetration (mm)	12.87			18.10			20.70		
Mass of wet soil + container (g)	51.99			53.79			74.66		
Mass of dry soil + container (g)	43.70			44.39			58.88		
Mass of container (g)	19.01			19.84			19.71		
Mass of moisture (g)	8.19			9.40			15.78		
Mass of dry soil (g)	24.69			24.55			39.17		
Moisture Content (%)	33.17			38.29			40.29		

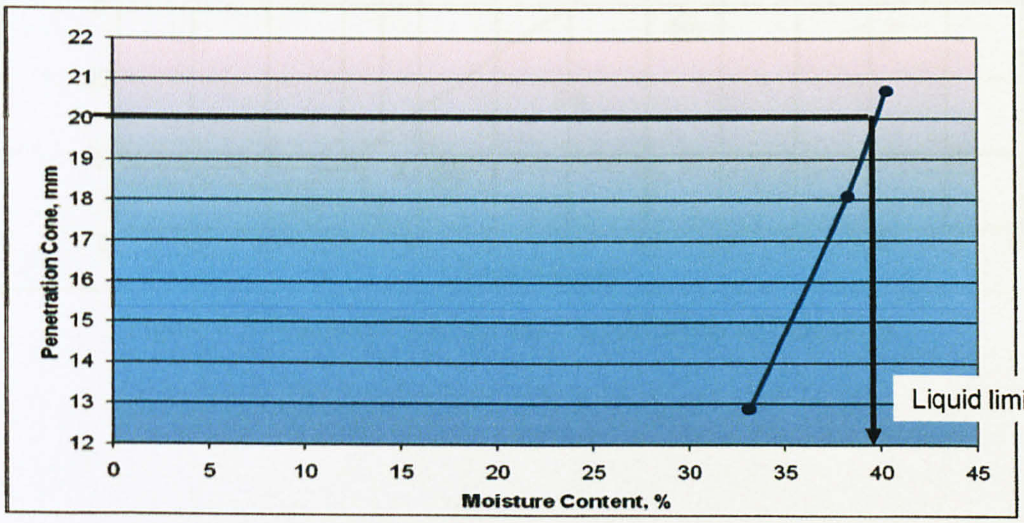


Figure 4.4 Graph of Cone Penetrometer Test

From the Figure 4.4 which is the graph of cone penetrometer test, when penetration is at 20 mm, the moisture content is 39.5% This value will be the liquid limit for the soil. Refering to the plastic limit value before which is 32.04 %, the plasticity index can be calculated.

Plasticity Index, Ip = LL – PL

where LL is the liquid limit value

PL is the plastic limit.

So Plasticity Index, Ip = 39.5 – 32.04

= 7.46 %

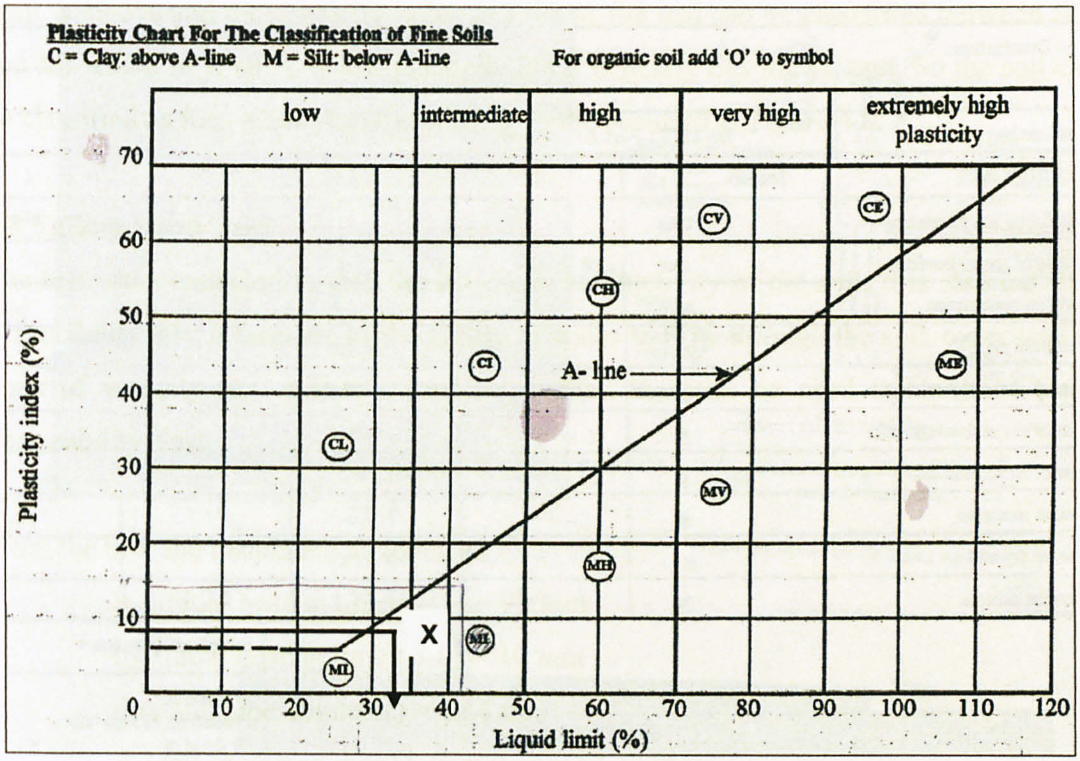


Figure 4.5 Plasticity Chart for the Classification of Fine Soils

	Primary letter	Secondary letter
Coarse-grained soils	G = GRAVEL S = SAND	W = well graded P = poorly graded Pu = uniform Pg = gap graded
Fine-gained soils	F = FINES (undifferentiated) M = SILT C = CLAY	L = low plasticity I = intermediate plasticity H = high plasticity V = very high plasticity E = extremely plasticity
Organic soils	Pt = PEAT	O = Organic

Figure 4.6 Sub group symbols in the British Soil Classification System

Referring to Figure 4.5; Plasticity Chart for the Classification of Fine Soils, when liquid limit is 39.5% and the plasticity index is 7.46 %, the soil can be classfified between ML and MI which M is silt L is low plasticity but it is nearly lies in ML part. So the soil can be classified as fine-grained soil according to the standard at Figure 4.6.

4.8 Falling Head Test

The test was conducted to find the hydraulic conductivity of the soil. It is also called a permeability test; a measure of the ability of water to flow through the soil, expressed in units of velocity (ex. cm/sec). Another method that can be used is Constant Head Permeability Test.

From the test, the results are as follows:

h_1 (initial head at time $t = 0$) = 90 mm

h_2 (initial head at time $t \neq 0$) = 10 mm

d_1(stand pipe diameter) = 4.64 mm

d_2 (soil specimen mould diameter) = 63.13 mm

L (soil specimen lenght) = 99.67 mm

Q (measured flow) = 10 ml

t (time) = 10 sec

To find q (rate of flow), the author use the formula;

$q = Q/t$

$= 10 \text{ ml} / 14 \text{ sec}$

$= 0.7143 \text{ ml/sec}$

To find a (area of the stand pipe), the author use the basic area formula which is;

$a = (\pi D^2) / 4$

$= \pi (4.64^2) / 4$

$= 16.91 \text{ mm}^2$

while the A (area of specimen mould) is also calculated with the same formula and the author get;

$A = (\pi D^2) / 4$

$= \pi (63.13^2) / 4$

$= 3130.12 \text{ mm}^2$

After have all the values, the permeability value can be calculated by the formula;

Permeability, $k = 2.303 \times (aL/At) \times (\log_{10}(h_1/h_2))$ in mm/sec

where h_1 is the initial head at time $t = 0$

h_2 is the initial head at time $t \neq 0$

d_1 is the stand pipe diameter

d_2 is the soil specimen mould diameter

L is the soil specimen length

a is an area of the stand pipe

A is an area of specimen mould

t is the time

Permeability, $k = 2.303 \times ((16.91 \text{ mm}^2 \times 99.67 \text{ mm})/(3130.12 \text{ mm}^2 \times 14)) \times \log_{10}(90/10)$

$= 0.0845 \text{ mm/sec}$

Convert the value into cm/sec and the value will be 0.00845 cm/sec. This value shows that the soil is fine sand which supposedly has k value from 0.001 – 0.01 cm/sec.

4.9 Constant Head Test

4.9.1 Without Soil Modification (Without Calcium Carbonate)

From the test, the results are as follows:

Flow of the water, Q = 135 mL

Length of the soil, L = 150 mm

Radius of the pipe, R = 2.8 inch = 71.12 mm

Height from the water head of inlet and water head of outlet, H = 22 mm

So from the formula of permeability, $K = QL/(\prod R^2H)$

$= 135(150) / (\prod (71.12)^2 (22)$

$= 0.0593$ mm/sec

Convert the value into cm/sec and the value will be 0.00593 cm/sec. This value shows that the soil is still fine sand which supposedly has k value from 0.01 – 0.001 cm/sec. It is quite differ from the falling head test may be because the error occurred during the test or etc.

4.9.2 With Soil Modification (Adding 5% of Calcium Carbonate)

From the test, the results are as follows:

Flow of the water, Q = 686.80 mL/sec

Length of the soil, L = 600 mm

Radius of the pipe, R = 8.0 inch = 200 mm

Height from the water head of inlet and water head of outlet, H = 35 mm

So from the formula of permeability, $K = QL/(\prod R^2H)$

$= 686.80 (600) / (\prod (200)^2 (35)$

$= 0.0937$ mm/sec

Convert the value into cm/sec and the value will be 0.00937 cm/sec. This value shows that the soil is still fine sand which supposedly has k value from 0.001 – 0.01 cm/sec. It is quite different from the falling head test may be because the error occurred during the test. By comparing the result with and without modification, it shows that the hydraulic conductivity, k is increased after modification even though the soil is still under the fine sand. By knowing this, it can conclude that the water will flow better in the modified soil.

4.10 Direct Shear Box Test

The test was conducted by using two samples of soils. The first sample is the sample from soil sampling area. The second sample is the sample after modification which is the soil with addition of 5% of Calcium Hydroxide (CaOH). In the direct test, a square prism soil is laterally restrained and shear along a mechanically induced horizontal plane while subjected to pressure applied normal to that plane. The shearing resistance offered by the soil as one portion is made to slide on the other is measured at regular intervals of displacement. Failure occurs when the shearing resistance the maximum value which the soil can sustain. By carrying out the tests on set (three) similar specimens of the same soil under different normal pressures, the relationship between measured shear stress at failure and normal applied is obtained.

Specimens are not fall along weakest plane but along predetermined or induced failure plane; horizontal plane separating the two halves of shear box. This is the main draw back of the test. During loading, the state of stress can only be evaluated at failure condition.

4.10.1 Direct Shear Box for Soil from Soil Sampling Area

a) Soil direct from field, Normal load (Vertical Normal Stress) = 100 kN/m^2

Table 4.9 Field Sample; Vertical Normal Stress = 100 kN/m^2

Time (min)	Force Gauge Reading, F (N)	Shear stress, F/A (kN/m^2)	Horizontal Displacement		Vertical Displacement
			Measured (mm)	Cumulative (mm)	Cumulative (mm)
2	232.46	23.246	0.86	0.86	0.03
4	411.73	41.173	1.81	2.67	0.03
6	539.78	53.978	1.56	4.23	0.03
8	646.16	64.616	2.71	6.94	0.03
10	715.11	71.511	0.84	7.78	0.03
12	764.36	76.436	1.94	9.72	0.03
14	784.06	78.406	1.82	11.54	0.01
16	786.03	78.603	1.94	13.48	0.01
18	786.03	78.603	194	15.42	0.01

b) Soil direct from field, Normal load (Vertical Normal Stress) = 200 kN/m^2

Table 4.10 Field Sample; Vertical Normal Stress = 200 kN/m^2

Time (min)	Force Gauge Reading, F (N)	Shear stress, F/A (kN/m^2)	Horizontal Displacement		Vertical Displacement
			Measured (mm)	Cumulative (mm)	Cumulative (mm)
2	204.88	20.488	0.96	0.96	0.02
4	577.21	57.721	1.4	2.36	0.02
6	734.81	73.481	1.88	4.24	0.02
8	829.37	82.937	1.7	5.94	0.02
10	1134.72	113.472	1.79	7.73	0.02
12	1199.73	119.973	1.99	9.72	0.02
14	1199.73	119.973	2.71	12.43	0.02
16	1205.64	120.564	1.35	13.78	0.02
18	1205.64	120.564	1.85	15.63	0.02

c) Soil direct from field, Normal load (Vertical Normal Stress) = 300 kN/m^2

Table 4.11 Field Sample; Vertical Normal Stress = 300 kN/m^2

Time (min)	Force Gauge Reading, F (N)	Shear stress, F/A (kN/m^2)	Horizontal Displacement		Vertical Displacement
			Measured (mm)	Cumulative (mm)	Cumulative (mm)
2	204.88	20.488	0.67	0.67	0.01
4	409.76	40.976	1.71	2.38	0.01
6	598.88	59.888	2.44	4.82	0.005
8	784.06	78.406	1.26	6.08	0.005
10	1032.28	103.228	1.75	7.83	0.005
12	1109.11	110.911	1.87	9.7	0.005
14	1207.61	120.761	1.18	10.88	0.005
16	1329.75	132.975	2.36	13.24	0.005
18	1363.24	136.324	1.52	14.76	0
20	1367.18	136.718	0.86	15.62	0

From the 4.9, 4.10 and 4.11, the graph of shear stress vs horizontal displacement are plotted in the Figure 4.7.

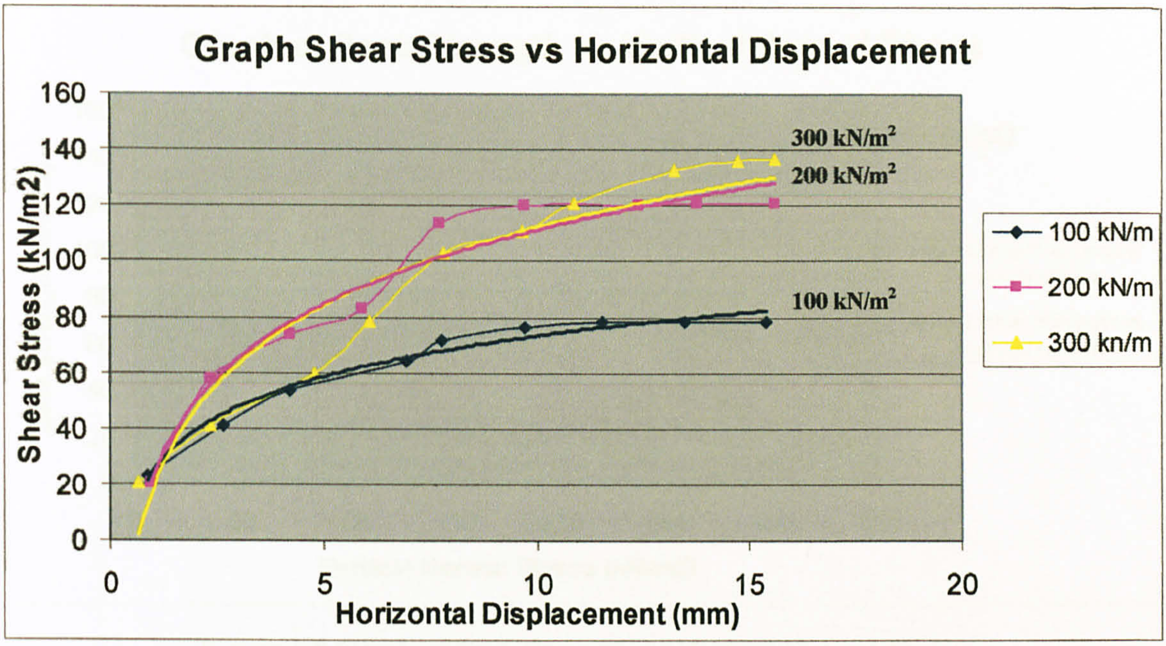


Figure 4.7 Graph of Shear Stress vs Horizontal Displacement

From each stress – displacement graph, the maximum value of shear stress and corresponding horizontal displacement is determined. Table 4.12 below shows the summary of peak strength, horizontal displacement and vertical displacement of the samples.

Table 4.12 Peak Strength, Horizontal Displacement and Vertical Displacement

Vertical Normal Stress (kN/m^2)	Peak Strength (kN/m^2)	Horizontal Displacement (mm)	Vertical Displacement (mm)
100	78.603	15.42	0.03
200	120.564	15.63	0.02
300	136.718	15.62	0.01

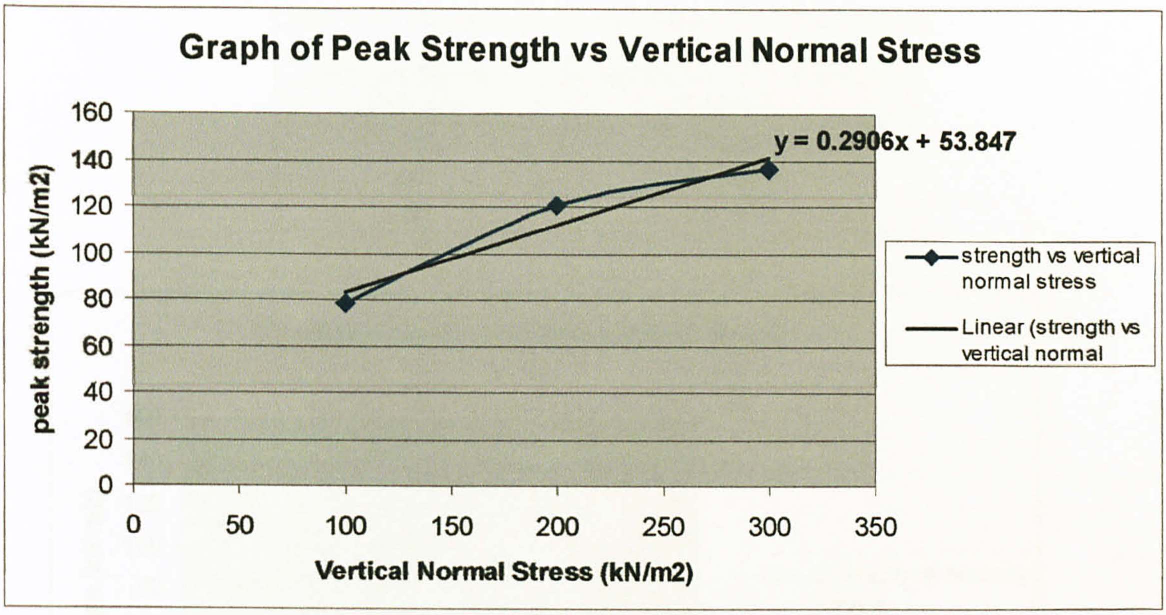


Figure 4.8 Graph of Peak Strength vs Vertical Normal Stress

From the strength – stress graph jshown in Figure 4.8;

1. Slope of graph gives the angle of friction, φ'
2. Intercept of the graph gives the apparent cohesion, c'

So, the angle of friction, φ' is 0.2906 while the apparent cohesion, c' is 53.847. By using shear strength equation of Coulumb 1776 below, the strength of the sample is determined.

Shear strength, $\tau = c' + \sigma' \tan(\varphi')$

where c' = apparent cohesion

σ' = normal stress on the failure plane

φ' = angle of friction

So, using the 100 kN/m^2, 200 kN/m^2 and 300 kN/m^2 normal stresses, the sample strength are as shown in Table 4.13. The strength vs normal stress graph of these samples are shown in Figure 4.8.

Table 4.13 Normal Stress and Strength of the sample

Normal Stress (kN/m^2)	Strength, $\tau = c' + \sigma' \tan(\varphi')$ (kN/m^2)
100	82.907
200	111.967
300	141.027

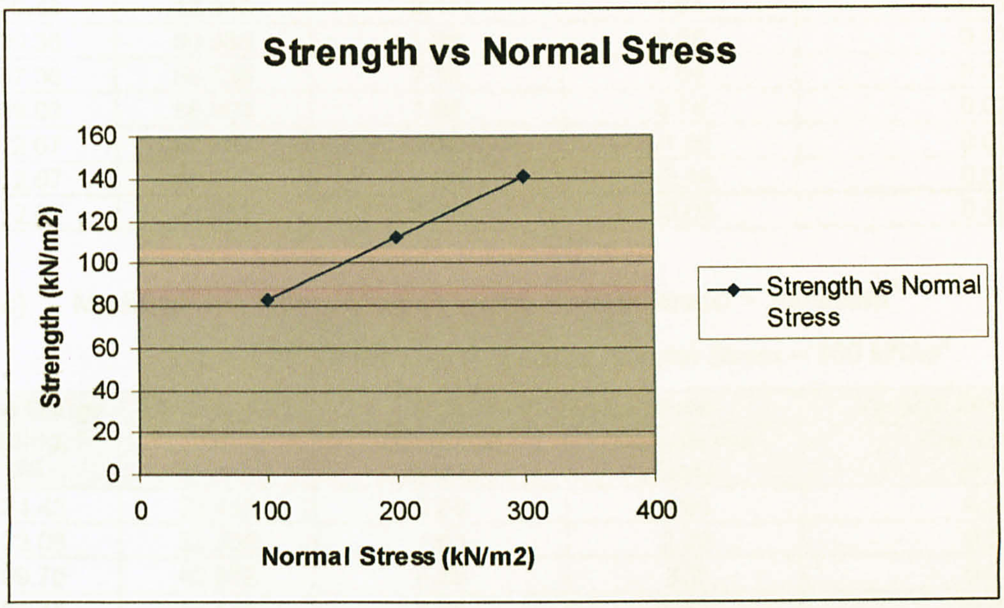


Figure 4.8 Strength vs Normal Stress for Direct Field Soil

4.10.2 Direct Shear Box for Modified Soil (Addition of 5% CaOH)

a) Modified soil, Normal load (Vertical Normal Stress) = 100 kN/m^2

Table 4.14 Modified soil; Vertical Normal Stress = 300 kN/m^2

Time (min)	Force Gauge Reading, F (N)	Shear stress, F/A (kN/m^2)	Horizontal Displacement		Vertical Displacement
			Measured (mm)	Cumulative (mm)	Cumulative (mm)
2	165.48	16.548	0.83	0.83	0.03
4	260.04	26.004	0.86	1.69	0.03
6	311.26	31.126	1.54	3.23	0.03
8	344.75	34.475	1.09	4.32	0.03
10	362.48	36.248	2.98	7.3	0.03
12	372.33	37.233	1.94	9.24	0.03
14	372.33	37.233	0.94	10.18	0.01
16	370.36	37.036	3.71	13.89	0.01

b) Modified soil, Normal load (Vertical Normal Stress) = 200 kN/m^2

Table 4.15 Modified soil; Vertical Normal Stress = 200 kN/m^2

Time (min)	Force Gauge Reading, F (N)	Shear stress, F/A (kN/m^2)	Horizontal Displacement		Vertical Displacement
			Measured (mm)	Cumulative (mm)	Cumulative (mm)
2	208.82	20.882	0.63	0.63	0.02
4	342.78	34.278	0.9	1.53	0.02
6	429.46	42.946	2.41	3.94	0.02
8	500.38	50.038	1.64	5.58	0.02
10	567.36	56.736	2.36	7.94	0.02
12	589.03	58.903	1.84	9.78	0.02
14	612.67	61.267	1.75	11.53	0.02
16	612.67	61.267	1.91	13.44	0.02
18	612.67	61.267	6.59	20.03	0.02

c) Modified soil, Normal load (Vertical Normal Stress) = 300 kN/m^2

Table 4.16 Modified soil; Vertical Normal Stress = 300 kN/m^2

Time (min)	Force Gauge Reading, F (N)	Shear stress, F/A (kN/m^2)	Horizontal Displacement		Vertical Displacement
			Measured (mm)	Cumulative (mm)	Cumulative (mm)
2	234.43	23.443	0.69	0.69	0.03
4	323.08	32.308	1.63	2.32	0.03
6	409.76	40.976	0.88	3.2	0.03
8	561.45	56.145	2.33	5.53	0.03
10	634.34	63.434	1.9	7.43	0.03
12	679.65	67.965	2.27	9.7	0.03
14	679.65	67.965	2	11.7	0.01
16	681.62	68.162	1.4	13.1	0.01
18	681.62	68.162	1.24	14.34	0.01

From the Table 4.14, 4.15 and 4.16, the graph of shear stress vs horizontal displacement are plotted as shown in Figure 4.9.

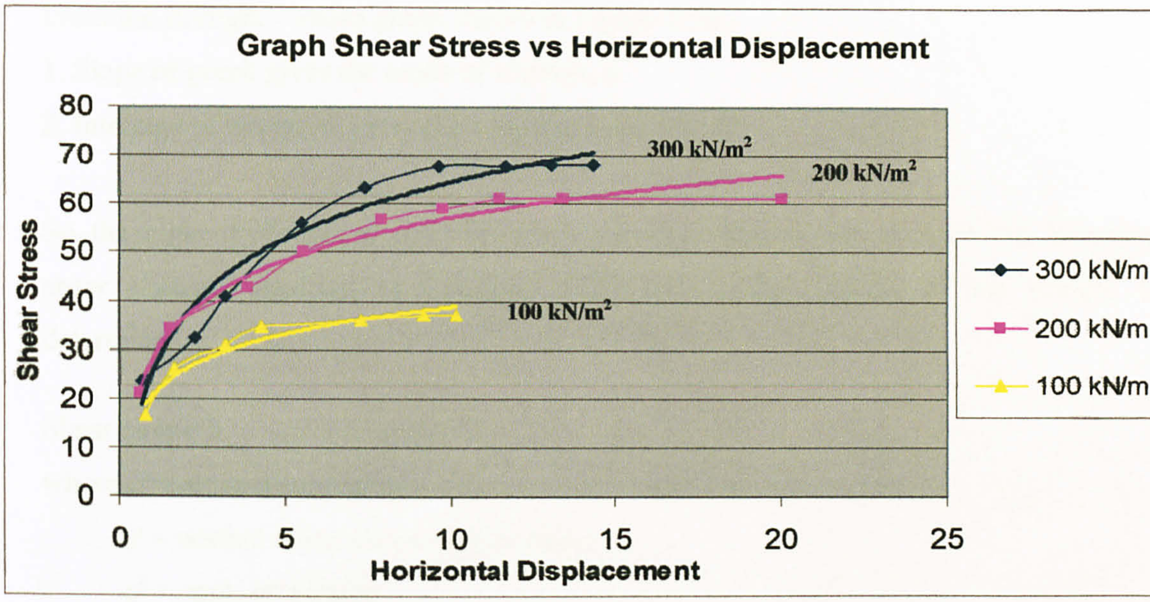


Figure 4.9 Graph of Shear Stress vs Horizontal Displacement

From each stress – displacement graph, the maximum value of shear stress and corresponding horizontal displacement is read-off. Therefore,

Table 4.17 Peak Strength, Horizontal Displacement and Vertical Displacement

Vertical Normal Stress (kN/m^2)	Peak Strength (kN/m^2)	Horizontal Displacement (mm)	Vertical Displacement (mm)
100	37.036	13.89	0.03
200	61.267	20.03	0.02
300	68.162	14.34	0.01

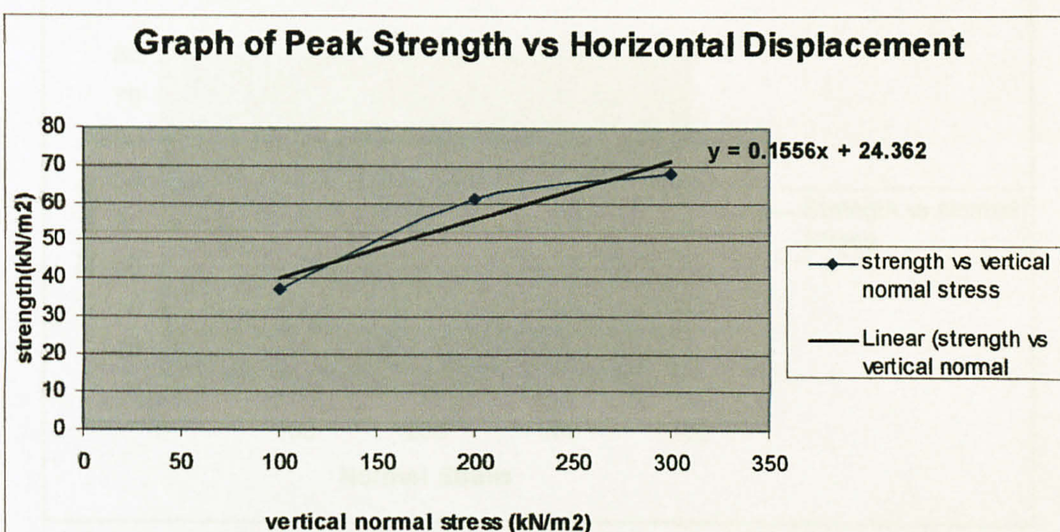


Figure 4.10 Graph of Peak Strength vs Vertical Normal Stress

From the strength – stress graph shown in Figure 4.10;

1. Slope of graph gives the angle of friction, φ'
2. Intercept of the graph gives the apparent cohesion, c'

So, the angle of friction, φ' is 0.1556 while the apparent cohesion, c' is 24.362. By using shear strength equation of Coulumb 1776 below, the strength of the sample is determined.

Shear strength, $\tau = c' + \sigma' \tan(\varphi')$

where c' = apparent cohesion

σ' = normal stress on the failure plane

φ' = angle of friction

So, using the 100 kN/m^2, 200 kN/m^2 and 300 kN/m^2 normal stresses, the sample strength are:

Table 4.18 Normal Stress and Strength of the sample

Normal Stress (kN/m^2)	Strength, $\tau = c' + \sigma' \tan(\varphi')$ (kN/m^2)
100	39.922
200	55.482
300	71.042

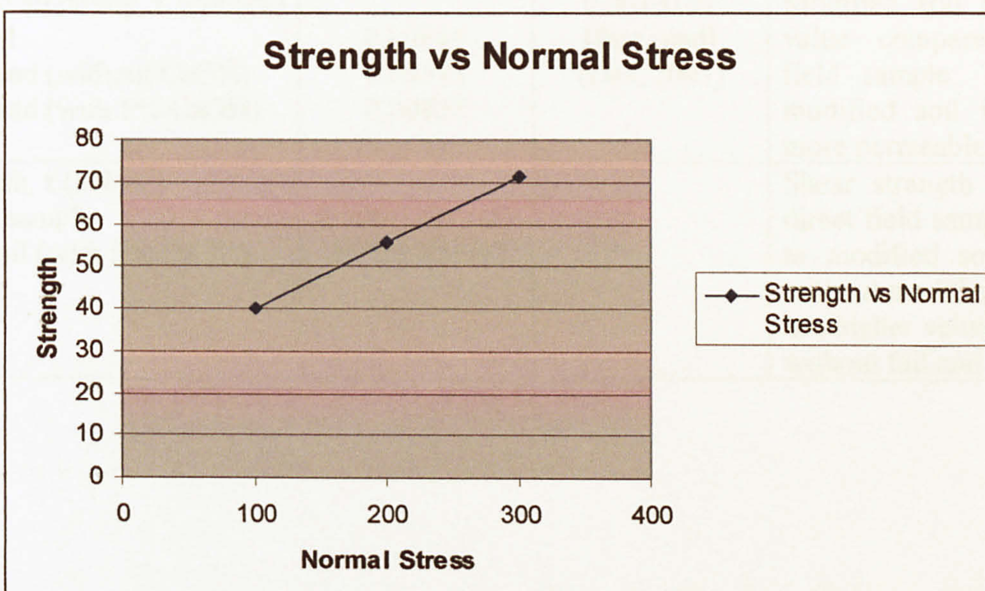


Figure 4.10 Strength vs Normal Stress for Modified Soil

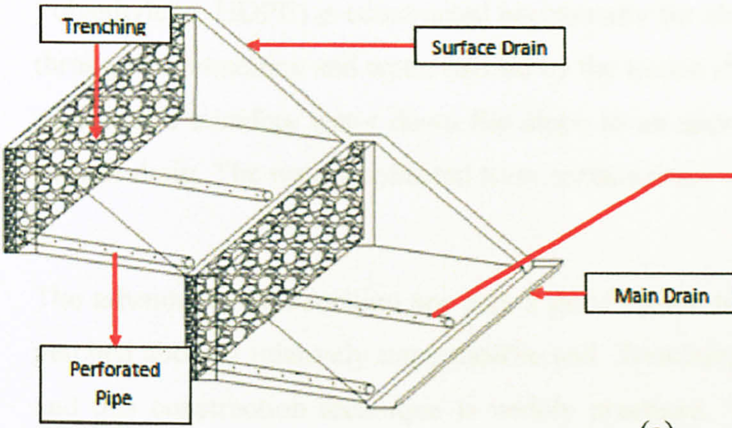
So, using the 100 kN/m^2, 200 kN/m^2 and 300 kN/m^2 normal stresses, the sample strength are as shown in Table 4.18. The strength vs normal stress graph of these samples are shown in Figure 4.10. From both result, it shows that the strength of modified soil is lesser than the direct field soil. Even though from the hydraulic conductivity test we can conclude that modified soil has higher k value, but it has lesser strength value compared to direct field soil (without modification).

Here is the summary of all soil properties and characteristics from laboratory testing.

Table 4.19 Summary of Soil Properties and Characteristics from Laboratory Testing

Test	Results	Standard	Remarks
Moisture Content (%) -Direct field sample -Modified Soil	 17.75 17.79	-	Under normal condition
pH Value	4.71	-	Acidic soil (pH < 7.0)
Specific Gravity, S	2.655	2.6-2.7 (Das, 2001)	Fine, light-colored sand
Particle Size Distribution -Sieve Analysis -Hydrometer Test	Fine soil	BS 1377:Part 2 From Semi-logarithmic paper (fine soil)	Most of particles found in fine siever (size 63 μm – 2 mm) and also has silt particle.
Plasticity Index, Ip -Plastic limit -Liquid limit	7.46 % 32.04% 39.50%	-	From plasticity chart for fine soil: ML (silt with low plasticity)
Hydraulic Conductivity, k (cm/sec) -Falling Head -Constant Head (without CaOH) -Constant Head (with 5% CaOH)	 0.00845 0.00593 0.00937	0.001-0.01 [fine sand] (Das, 2001)	Modified soil has higher k value compared to direct field sample. The flow in modified soil is better and more permeable.
Shear Strength, τ (kN/m^2) -Direct field sample -Modified Soil (with 5% CaOH)	 82.907-141.027 39.922-71.042	-	Shear strength is higher in direct field sample compared to modified soil. It means, without modification, the soil has higher value to resistance without fail and slide.

4.11 Design of Trenching



(a)

(b)

Figure 4.11 (a) Proposed Design of Trenching at Actual Slope; (b) Example of cross section of the trenching

Figure 4.11 above shows the proposed design of trenching at actual slope or actual application. The design includes the trenching, surface drain, perforated pipes and main drain which carry different function. This is not the real scale of the design because it is only proposed design and it scale out depends on the height and area of the slope. As for engineering consideration, the terrace here should be around 1.5m width, including trenching area because it depends on the excavator size that will dig the trenches. The trenching is dug along the proposed future back alignment of the slope and width about 24 inches or 60 cm. Trenching is suitable to shallow groundwater less than 10 feet below slope crest. The trenching is filled with gravel vertically for about 4 to 6 feet depth; depends on how deep the excavator can dig. It is supposedly excavated into a relatively impermeable soil layer and installed to collect and remove groundwater as it flows across the impermeable layer. It can be excavated with curves and bends to prevent cutting tree roots and hitting underground utilities. Trenches can be covered with topsoil and replanted to conform to the existing ground conditions.

Trenching is then lined with a quality geotextile that does not clog. The main function of the trenching is to prevent the groundwater to flow directly to the slope; so the slope will not easily fail and the groundwater will not affect the stability of the slope. In addition, the perforated pipes are installed to become a medium to flow the water from to the surface drain before drain out to main drain. Perforated pipe (may be from High Density Polyethylene, HDPE) is constructed horizontally for about 1.5m. Groundwater will flow through the trenching and water carried by the trench should be conveyed to a perforated pipe which transfers water down the slope to an appropriate discharge point which is surface drain. The waters collected from surface drain will drain out to main drain.

The advantages of trenching are it is a good option to intercept groundwater which is perched above a relatively impermeable soil. Trenching also a good coverage technique and this construction technique is widely practiced. Trenching also has disadvantage which is pipes in the system are often undersized. If not properly backfilled and compacted, surface water can flow into trench and cause additional problems for the slope and drainage system.

From the proposed design at the actual slope, the author made a model in the laboratory to evaluate the effectiveness of this method as a slope protection in granite soil by determining the moisture content in the slope and the clarity of water drain out from the perforated pipes.Figure 4.12 shows the diagram of the design and model of the trenching

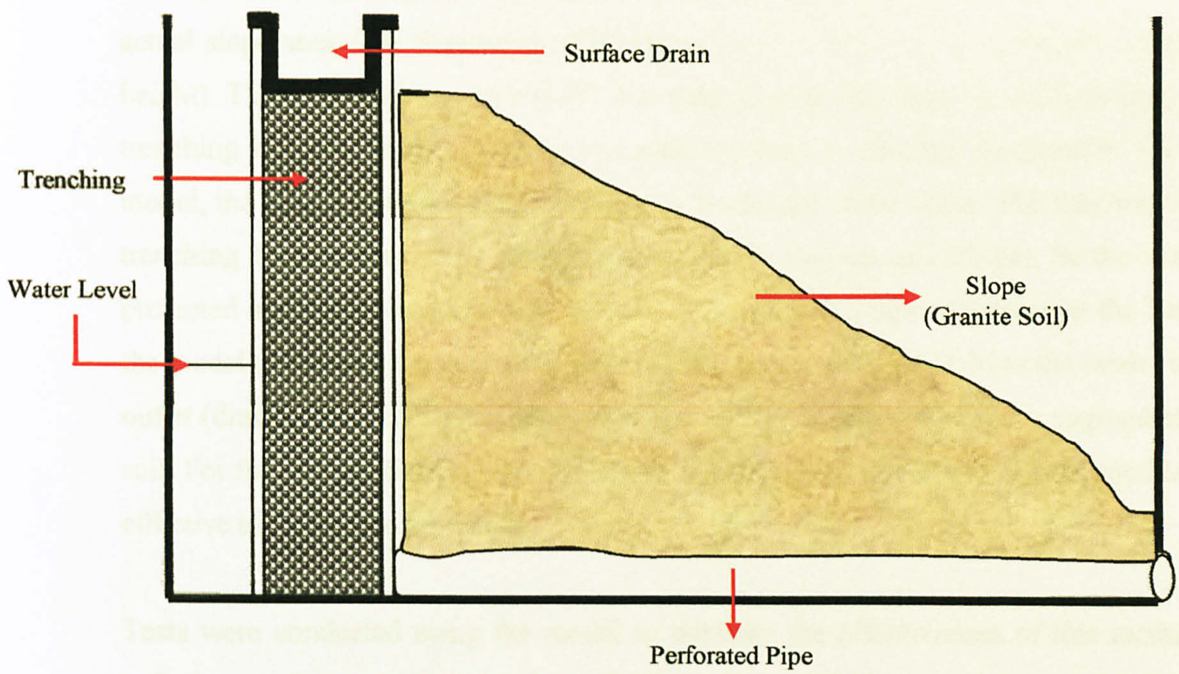


Figure 4.12 (a) Design of Trenching (Laboratory)

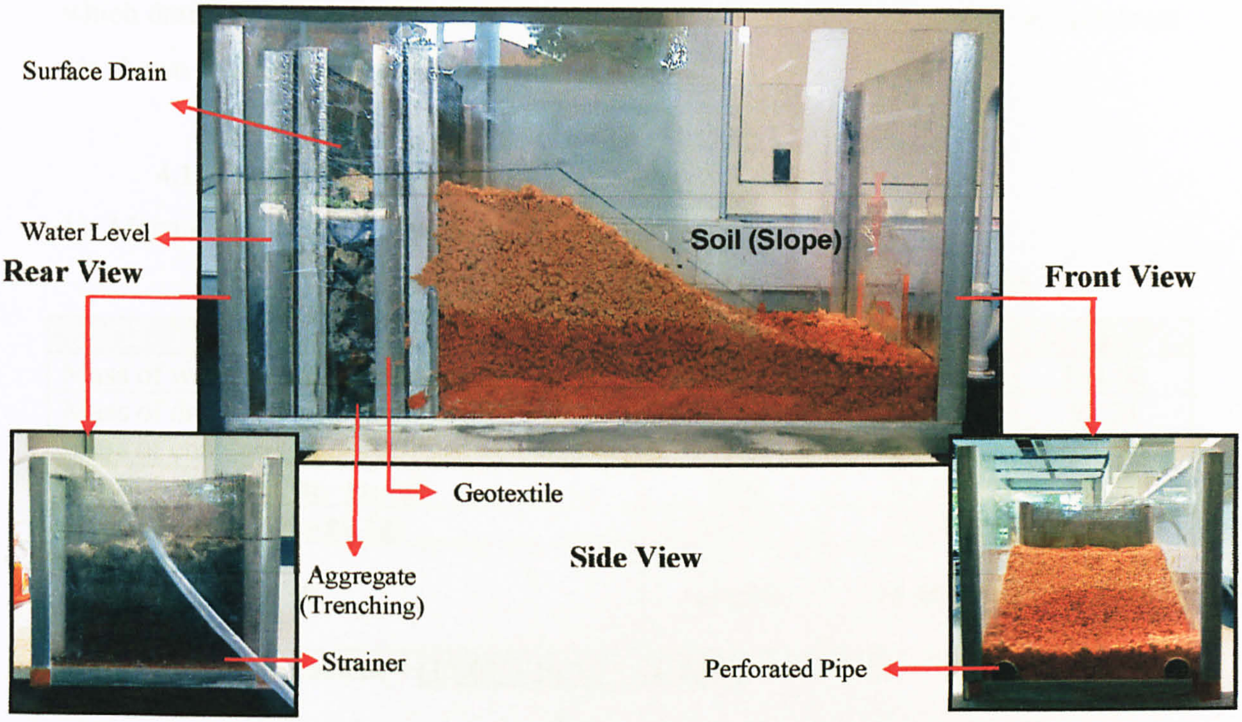


Figure 4.12 (b) Model of Trenching (Laboratory)

The model is used only for laboratory experiment and not using the real scale with the actual slope area. The dimension of the whole model is 2' x 1' x 1' (length x width x height). The height of the slope is 8". The main part of the model is the trenching. The trenching is made from aggregate to replace gravel in the real application. For this model, the length of trenching is one fifth of the length of the slope. The function of the trenching is to prevent water from flowing directly into the soil (slope). So the slope is protected and failure will not easily occur. The perforated pipe is located at the base of the model to become the horizontal drain. It will channel the water from the model to the outlet (drain). The geotextile and strainer are used as separators between aggregates and soil. For the real application, the dimension is determined to make sure it is suitable and effective to be a slope protection.

Tests were conducted using the model to evaluate the effectiveness of this method in reducing moisture content in the soil. Oven-dry method was used to determine the moisture content in the slope from three different locations and see the clarity of water which drains out from the perforated pipes. The results of moisture content at each level are shown in Table 4.20, 4.21 and 4.22.

4.11.1 Moisture content

1) Moisture content at the surface of the slope (H=20 cm):

Table 4.20 Moisture content at the surface of the slope

Container No.	1	2	3
Mass of wet soil + container, m_2 (g)	89.30	106.55	104.02
Mass of dry soil + container, m_3 (g)	80.06	94.44	92.24
Mass of container, m_1 (g)	18.60	20.68	20.50
Mass of moisture, m_2-m_3 (g)	9.24	12.11	11.78
Mass of dry soil, m_3-m_1 (g)	61.46	73.76	71.74
Moisture content, $W = \frac{m_2-m_3}{m_3-m_1} \times 100\%$	15.03%	16.42%	16.42%

Average moisture content = $\frac{(15.03 + 16.42 + 16.42)\%}{3}$ = 15.96 %

2) Moisture content at the middle of the slope (H=11cm):

Table 4.21 Moisture content at the middle of the slope

Container No.	1	2	3
Mass of wet soil + container, m_2 (g)	90.27	68.42	95.20
Mass of dry soil + container, m_3 (g)	80.30	60.96	82.85
Mass of container, m_1 (g)	18.46	19.10	20.82
Mass of moisture, m_2-m_3 (g)	9.97	7.46	12.35
Mass of dry soil, m_3-m_1 (g)	61.84	41.86	62.03
Moisture content, $W = \frac{m_2-m_3}{m_3-m_1} \times 100\%$	16.12%	17.82%	19.91%

Average moisture content = $\frac{(16.12 + 17.82 + 19.91)\%}{3}$ = 17.95 %

3) Moisture content at the base of the slope (H=2cm):

Table 4.22 Moisture content at the base of the slope

Container No.	1	2	3
Mass of wet soil + container, m_2 (g)	121.98	133.56	140.37
Mass of dry soil + container, m_3 (g)	105.07	113.86	117.80
Mass of container, m_1 (g)	21.10	19.01	18.59
Mass of moisture, $m_2 - m_3$ (g)	16.91	19.70	22.57
Mass of dry soil, $m_3 - m_1$ (g)	83.97	94.85	99.21
Moisture content, $W = \frac{m_2 - m_3}{m_3 - m_1} \times 100\%$	20.14%	20.77%	22.75%

$$\text{Average moisture content} = \frac{(20.14 + 20.77 + 22.75)\%}{3} = 21.22\ \%$$

From the tables, it shows that the moisture content is high at the base, moderate at the middle and low at the surface of the soil in the model. This is because the water were drained at the base of the model. From the results, the value is smaller than the value of liquid limit (32.04%) of the direct sample. So the trenching allows the reduction of moisture content. Therefore, it is potentially become one of the effective methods of slope control.

Figure 4.13 Samples of moisture content test

4.11.2 Clarity of water

Another test conducted was clarity test. The water that drained out from the perforated pipe, flow to the outlet into a container. The clarity of the water can be seen in Figure 4.14. The water is clear and did not contain any soil (sediment). The amount of water that flow out from the model was high; most of the water was not trapped in the soil. Thus, it improve the stability of the slope.

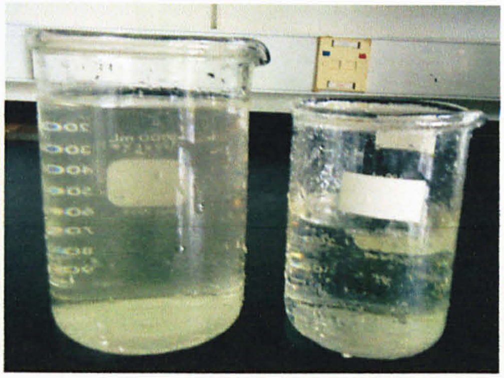


Figure 4.14 Water from the outlet of Trenching Model

4.12 Slope/W Analysis

The author analyse the factor of safety using Slope/W software by consider the situation before and after construct the trenching. The soil is a fine-grained soil with characteristics of silty sand. So the properties that used during the analysis are as follows:

- Typical saturated unit weight for the soil, $\gamma_{sat} = 19\ kN/m^3$
- Friction angle for the soil, $\varphi = 30^0$
- Cohesion, c = 2

The normal slope is assumed to be 5.0 m height. Assume that before putting the trench, the groundwater level is high, such as 4.0 m height. So the Figure 4.15 below shows the analysis if the safety factor when groundwater level is equal to 4.0 m and the possible location of failure when groundwater level is equal to 4.0 m height is shown in Figure 4.16.

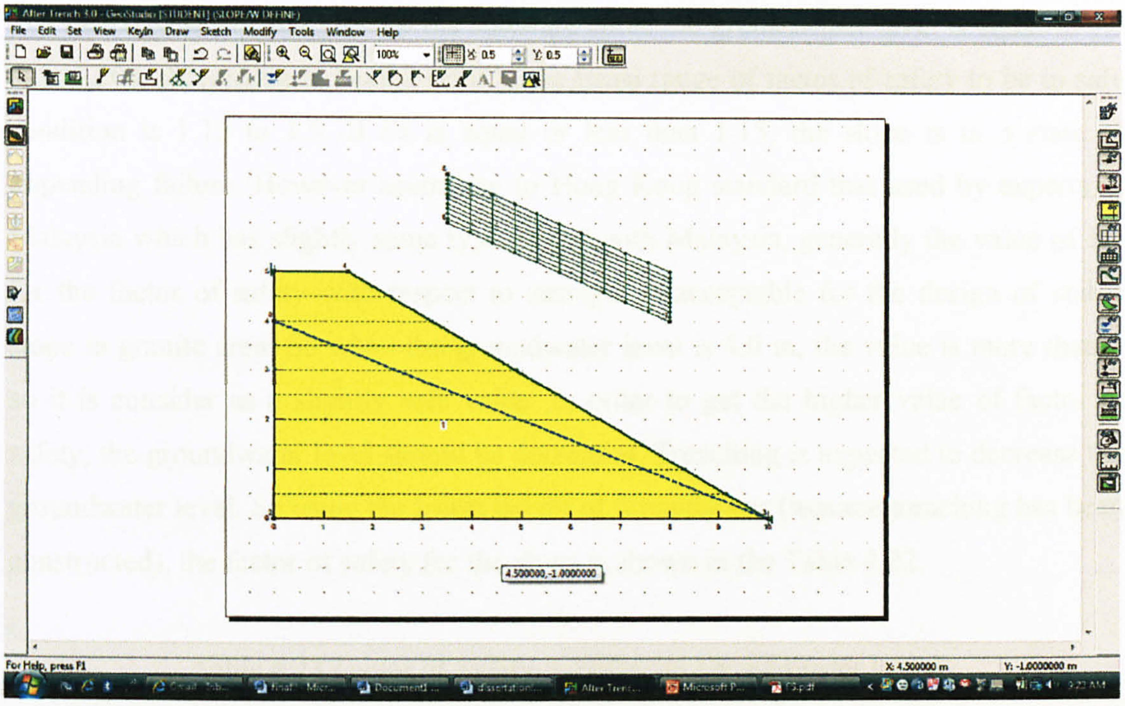


Figure 4.15 Analysis Slope/W when Groundwater Level = 4.0 m Height (Before Construct the Trenching)

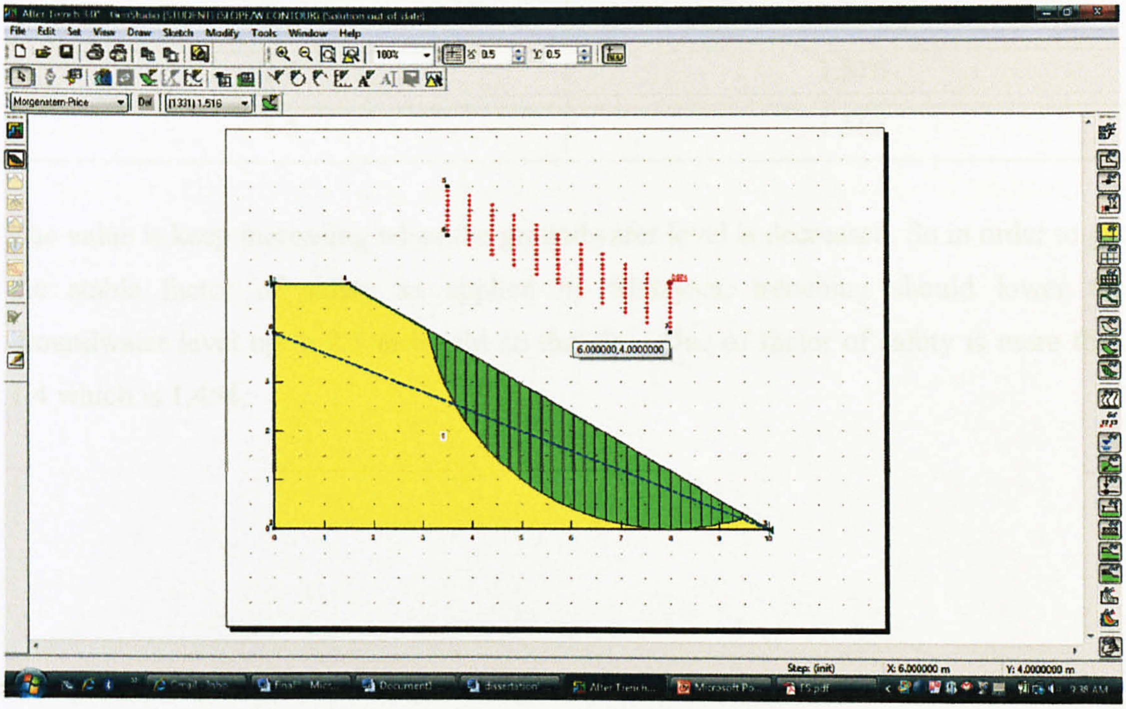


Figure 4.16 Possible Failure Location of the Slope

The factor of safety analysed by the software when the groundwater level is equal to 4.0 m is 1.204. According to Budhu (2007), the usual range of factor of safety to be in safe condition is 1.15 to 1.5. If Fs is equal or less than 1.15, the slope is in a state of impending failure. However according to Hong Kong standard that used by experts in Malaysia which has slightly same type of soil with Malaysia, generally the value of 1.4 for the factor of safety with respect to strength is acceptable for the design of stable slope in granite area. So when the groundwater level is 4.0 m, the value is more than 1 so it is consider as a slightly safe value. In order to get the higher value of factor of safety, the groundwater level should be decreased. Trenching is expected to decrease the groundwater level. So using the lesser height of groundwater (assume trenching has been constructed), the factor of safety for the slope is shown in the Table 4.22.

Table 4.23 Factor of Safety in Different Groundwater Level

Groundwater Level (m)	Factor of Safety
3.5	1.297
3.0	1.380
2.5	1.454
2.0	1.516
1.5	1.568

The value is keep increasing when the groundwater level is decreased. So in order to get the stable factor of safety as applied by Malaysia, trenching should lower the groundwater level up to 2.5 m height so that the value of factor of safety is more than 1.4 which is 1.454.

CHAPTER 5

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

Residual soil in granitic area consists of minerals from the weathering products of feldspar, quartz and mica minerals. The soil is a fine soil containing 17.75% of moisture content during normal condition, and has acidic pH value of 4.71. The specific gravity of granite soil is 2.655. The liquid and plastic limit tests indicate that the plasticity index of the soil is 7.46%. The content meaning the soil is silt with low plasticity (ML) according to the plasticity chart. The soil also has a hydraulic conductivity, k from 0.001 to 0.01 cm/sec which still under the standard of fine sand. In addition, from the shear box test, the shear strength of the soil from the study area is from 82.907 to 141.029 kN/m^2. The modification of the soil; by adding 5% of Calcium Hydroxide (CaOH) is not really affected the effectiveness of the water flow. Even though the modified soil has higher k value, immediate test shows that it has low shear strength. Trenching is an innovative technique to construct a protective trench at the slope area; to stabilize the slope from slope failure such as landslides. From the tests conducted to the trenching model, it shows that the moisture content of the soil (15.96-21.22%) is less than the liquid limit (39.50%). It means the trenching lowers the moisture level. The water that flow out from the perforated pipe is also clear and without sediment. From the Slope/W analysis, the factor of safety determined is 1.454 in order to get the stable factor of safety as applied by Malaysia standard. This control method is considered to be cost effective because the materials to be used are confined to the trench only.

5.2 Recommendation

There are some recommendations or suggested future work for expansion and continuation. Among the suggested work are:

i. Rescale the design for the real construction so that trenching can be applied with the correct and suitable scale.

ii. Continue the analysis by Slope/W software to computing the factor of safety of earth and rock slopes that can analyze problems of slip surface shapes, pore-water pressure conditions, analysis methods and loading conditions. From here, the author can do adjustment to the design in order to get more effective design.

CHAPTER 6

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APPENDICES

1.0 LABORATORY PROCEDURE:

a) Oven Drying Method

1 The moisture content tins has been cleaned and weigh to the nearest 0.01 g; known as m_1

2 Three samples of about 30 g placed in the three different tins and weigh to the nearest 0.01 g as m_2.

3 The sample then placed in the oven at 105^0 C for a period of 24 hours. After 24 hours, the samples have taken out from the oven and weigh it as m_3.

4 The moisture content then be calculated with the given formula

b) pH Test

1. Take 30 g of soil and put in the beaker. Dilute the soil with 75 ml distilled water and soak them together.
2. Leave for about 8 hours. Take the pH reading by using pH meter.

c) Particle Density or Specific Gravity Test

Specimen Reference: 1. Mass of jar + gas jar + plate + soil + water (m_3)

2. Mass of jar + gas jar + plate + soil (m_2)
3. Mass of jar + gas jar + plate + water (m_4)
4. Mass of jar + gas jar + plate (m_1)

d) Particle Size Distribution Test

1. The oven dried sample weighed to 500 g
2. Test sieve from size 2 mm until 63 μm stacked on the mechanical shaker appropriately.
3. The sample placed on the top sieve and the sieve was covered by the lid. The test sieve is agitated on the shaker for 10 minutes. The amount retained on each sieves weighed to 0.01% to its total mass.

e) Determination of Plastic Limit Test

1. The sample of 20 g of soils which passes 425 μm test sieve taken and placed on the glass plate
2. Allow the soil to wet partially on the plate until it becomes plastic enough to be shaped onto ball.
3. The ball moulded between the fingers and rolled between the palm of hand until the heat of hand had dried the soil sufficiently for slight cracks to appear on its surface. The sample divided into 4 groups and moulded them in fingers to equalize the distribution of moisture content, then the soil formed into a thread about 6 mm diameter
4. The thread rolled between the fingers. Picked up the soil and transfer them to the container and the soil then put into the oven dry.
5. The plastic limit calculated using the formula and followed by the calculation of plasticity index.

f) Sedimentation by the Hydrometer Method

B. Calibration

1. Determine the volume of hydrometer, Vh by weigh it to the nearest 0.1 g.
2. Scale calibration of hydrometer by measure the distance from the 100 mL scale marking to the 1000 mL scale marking on the sedimentation cylinder.
3. Measure the distance from the lowest calibration mark on the stem of the hydrometer to each of the major calibration marks, Rh.
4. Measure the distance, H that corresponding to a reading Rh is equal to the sum of the distances measured in A.2 and A.3 ($N+d_1$, $N+d_2$,...)
5. Measure and record the distance, h from the neck to the bottom of the bulb as the height of the bulb.
6. Calculate the effective depth, Hr (mm) corresponding to each of the major calibration marks, Rh from the equation $Hr = H + \frac{1}{2} [h - V_hL/900]$
7. Plot the relationship between Hr and Rh as a smooth graph

C. Preparation and Assembly

1. Weigh 50 g of the test sample and obtain the initial mass, m_0
2. Place the test sample in the wide mouth clonical flask
3. Add 100 mL of the sodium hexametaphosphate solution to the soil in the clonical flask and shake until all soil in suspension
4. Transfer all the material retained on 63 μm test sieves to an evaporating dish and add to measuring cylinder

D. Sedimentation

1. Insert rubber bung into the cylinder containing the soil suspension, shake and place it in the constant temperature bath so that it is immersed in water at least up to the 1 L gradation mark.
2. Add 100 mL of sodium hexametaphosphate to the second 1 L sedimentation cylinder and dilute with distilled water to exactly 1 L. insert the rubber bung and place this cylinder in the constant temperature bath alongside the first.
3. After at least 1 hour, take out the cylinder containing the dispersion solution and shake thoroughly and replace it in the bath. Take out the cylinder containing the soil suspension, shake it vigorously end-over-end about 60 times and immediately replace it in the bath.
4. Start the timer and remove the rubber bungs carefully from the cylinder.
5. Immerse the hydrometer in the suspension to a depth slightly below its floating position and allow it to float freely.
6. Take the reading at upper rim of the meniscus after period of 0.5 minute, 1 minute, 2 minute and 4 minute.
7. Remove the hydrometer slowly, rinse in distilled water and place it in the cylinder of distilled water with dispersion at same temperature as the soil suspension. Observe and record the top of the meniscus reading, Ro.
8. Reinsert the hydrometer in the soil suspension and take record readings after period of 8 minute, 30 minute, 2 hour, 8 hour and 24 hour from the start of sedimentation. Observe and record the temperature of the suspension once during the first 15 minute and after every subsequent reading. Read the temperature.

g) Direct Shear Box Test

1. Place or pour the sand directly into the assembled shear box from the quantity of known mass.
2. Level the surface of the specimen using a suitable template to give a specimen of the appropriate thickness, without disturbing the main body of the placed soil.
3. Place the porous plate on the specimen. Measure the height of the sample
4. Place the top spacer plate, and the loading cap carefully on top of the porous plate.
5. Start the test, record reading of the force measuring device, the horizontal displacement gauge, the vertical deformation gauge and elapsed time at regular intervals of horizontal displacement.
6. Repeat the steps for at least 3 determinations into the sample with addition of normal load from each test. The normal loadings are 100 kN/m^2, 200 kN/m^2 and 300 kN/m^2.